MERRIMACK VALLEY FLOOD CONTROL

DEFINITE PROJECT REPORT

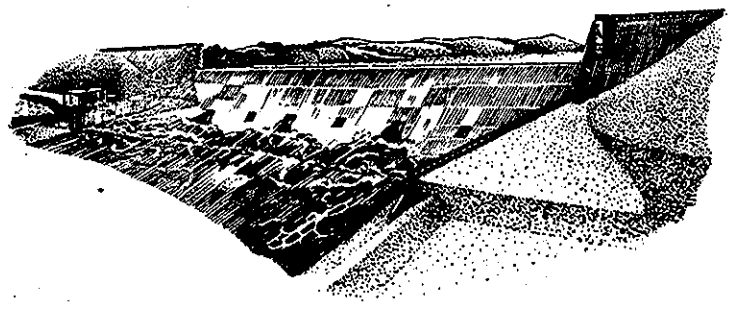
ON

BENNINGTON RESERVOIR

CONTOOCOOK RIVER

NEW HAMPSHIRE

APRIL-1945



CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE BOSTON, MASS.

407-825

DEFINITE PROJECT REPORT

ON

BENNINGTON RESERVOIR

CONTOOCOOK RIVER

NEW HAMPSHIRE

PREPARED IN THE U.S. ENGINEER OFFICE,

BOSTON, MASSACHUSETTS DATED APRIL 1945

APPROVED BY THE CHIEF OF ENGINEERS 1945

E.D. FILE _____

DATE	NEW PAGES OR DRAWINGS	DATE APPROVED BY C. OF E.

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

MERRIMACK VALLEY FLOOD CONTROL

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SECTION A.

PERTINENT DATA

DEFINITE PROJECT REPORT
BENNINGTON RESERVOIR

SECTION A - PERTINENT DATA

Following is a tabular summary of pertinent data for the construction of the initial stage of the proposed Bennington Reservoir, Merrimack River Basin, New Hampshire, as outlined in this project report:

1. Project Location . . . . . Contoocook River; Approximately 18 miles northeast of Keene, N. H., and 0.5 mile south of Bennington, N.H.: approximately at river mile 147 above the mouth of the Merrimack River.

2. Reservoir Data

Net drainage area	128 sq. miles
Storage capacity, inches on net drainage area	8.8
Gross drainage area	186 sq. miles
Storage capacity, inches on gross drainage area	6.0
Storage capacity for flood control	60,000 acre-feet
Reservoir area at elevation of spillway crest:	
Wooded land, containing thin stand of mixed hardwoods and some soft woods (25%)	970 acres
Wooded land, containing fair stand of mixed hard and soft woods, small amount merchantable (53%)	2060 acres
Cleared land - building lots, tillage and pasture (12%)	465 acres
Water area (10%)	390 acres
Total Reservoir Area (100%)	3885 acres
Length of Reservoir	7.5 miles
Maximum wave fetch	5.5 miles

3. Stream Flow Data

Average yearly flow	300 c.f.s.
Average annual flood	3,100 c.f.s.
Flood of 5 November 1927 (estimated)	5,500 c.f.s.
Flood of 19 March 1936	13,600 c.f.s.
Flood of 21 September 1938	15,400 c.f.s.
Spillway design flood at Bennington	64,000 c.f.s.
Spillway design flood - inflow to reservoir	77,600 c.f.s.

4. Dam

Type	Rolled earth fill
Elevation of Top of Dam	724.0 feet M.S.L.
Total Length of Dam	3,520 feet
Maximum Height of Earth Embankment	64 feet
Top Width of Earth Embankment	25 feet
Freeboard	7.2 feet
Power Development	None

5. Outlets

Location	In spillway section
Number of gates	6
Type of Gates	Hydraulically operated sluice gates
Size of Gate Openings	4'-0" x 6'-0"
Invert Elevation of Gates	667.0 feet M.S.L.
Time of emptying (80% of Capacity)	12 Days
Downstream Channel Capacity	4,000 cubic feet per second
Design Discharge with water surface at spillway crest, Elevation 705:	
1 Gate	920 cubic feet per second
6 Gates	5,520 cubic feet per second

6. Spillway

Type of Spillway	Concrete gravity ogee
Crest Elevation	705.0 Feet M.S.L.
Length	300 Feet
Spillway Design Flood Discharge	45,900 cubic feet per second
Maximum Water Surface Elevation	716.8 Feet M.S.L.

7. Foundations

Dam, General	The Dam is founded on impervious glacial till except a portion of the west embankment from Station 28 + 15 to the end at Station 36 + 90, which is founded on silty sand.
Non-overflow Sections	Impervious glacial till
Spillway	Impervious glacial till
Stilling Basin Retaining Walls	Impervious glacial till

8. Quantities

Embankment:	
Impervious Fill	148,000 cubic yards

8. Quantities (Cont'd.)

Pervious Fill	200,000 cubic yards
Random Fill	195,000 cubic yards
Semicompacted Fill	177,500 cubic yards
Special and Processed Material	104,500 cubic yards
Rock Slope Protection	90,500 cubic yards
Total Embankment	915,500 cubic yards
Excavation	654,000 cubic yards
Concrete:	
Spillway	23,800 cubic yards
Non-overflow Sections	12,600 cubic yards
Stilling Basin	8,600 cubic yards
Stilling Basin Walls	7,800 cubic yards
Total Concrete	52,800 cubic yards

9. Estimated Cost

Reservoir Clearing	$ 4,000
Reservoir Costs	1,482,000
Construction Costs	2,514,000
Total	$ 4,000,000
Cost per Acre Foot of Total Storage	$ 66.67

SECTION B.

SYLLABUS

DEFINITE PROJECT REPORT
BENNINGTON RESERVOIR

SECTION B - SYLLABUS

The Bennington Reservoir was approved by the Chief of Engineers on 10 December 1943 and is a part of the comprehensive plan for flood control of the Merrimack River Basin as authorized by the Flood Control Act approved 22 June 1936 and amended by the Flood Control Act approved 28 June 1938.

In the proposed initial flood control development of the Bennington site the dam consists of a concrete spillway, concrete non-overflow sections, and earth filled embankment sections. The concrete spillway is a wide-base gravity type ogee section with a crest length of 300 feet and a height of 49 feet from the stilling basin floor to the crest. Six gated outlets, each 4'-0" x 6'-0", with operating chambers reached by means of a passageway, are located in the spillway section for the regulation of water flow. The concrete non-overflow sections are adjacent to the spillway and consist of gravity sections 97'-6" long which connect with the earth embankment. The embankment section has a length of 3,220 feet and a maximum height of 64 feet. It is constructed of rolled earth fill with a central impervious core and is protected by an upstream rock blanket and downstream rock-filled drainage toe. All masonry structures and the impervious core of the embankment extend to or into the impervious glacial till foundation except a portion of the core of the west abutment which is founded on silty sand. A series of wells are provided on the downstream side of the embankment within this area to relieve the hydrostatic head. The stilling basin is composed of a concrete mat with reinforced concrete baffles, end sill and concrete gravity-type guide walls. In view of the favorable results of an analysis of this project for future conservation storage in addition to flood control storage, provision has been made in the initial design for an ultimate addition of 7 feet to the spillway and 6 feet to the embankment. The estimated cost for construction of the dam over a period of two years is $4,000,000. which includes the reservoir and construction costs. Local financial co-operation is not required for this project. Authorization for preparation of this project report is contained in letter from the Chief of Engineers, Washington, D.C., to the Division Engineer, New England Division, dated 18 December 1943, subject: "Definite Project Reports for Bennington and Beards Brook Reservoirs," OCE File No. CE 821.2 (Hopkinton-Everett Dam).

SECTION C.

TEXT OF REPORT

War Department
United States Engineer Office
Boston, Massachusetts

MERRIMACK RIVER BASIN FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT
BENNINGTON RESERVOIR
CONTOOCOOK RIVER, N. H.

SECTION C - TEXT OF REPORT
April 1945

1. Project Authorization.- The Bennington Reservoir for flood control protection in the Contoocook River Basin, New Hampshire, as described herein, is proposed as an element of the comprehensive plan for flood control reservoirs and related flood control work for the Merrimack River Basin authorized by the following portions of the Flood Control Acts of 1936 and 1938.

a. Flood Control Act, approved 22 June 1936 (Public - No. 738 - 74th Congress):

"FLOOD CONTROL ACT OF 1936

"Sec. 5. That pursuant to the policy outlined in sections 1 and 3, the following works of improvements, for the benefit of navigation and the control of destructive flood waters and other purposes, are hereby adopted and authorized to be prosecuted, in order of their emergency as may be designated by the President, under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with the plans in the respective reports and records hereinafter designated: Provided, That penstocks or other similar facilities, adapted to possible future use in the development of adequate electric power may be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers.

****

"MERRIMACK RIVER, NEW HAMPSHIRE AND MASSACHUSETTS

"Construction of a system of flood-control reservoirs in the Merrimack River Basin for the reduction of flood heights in the Merrimack Valley generally; estimated construction cost, $7,725,000; estimated cost of lands and damages, $3,500,000."

b. Flood Control Act, approved 28 June 1938 (Public No. 761 - 75th Congress, 3rd Session):

> "Sec. 4. That the following works of improvement for the benefit of navigation and the control of destructive floodwaters and other purposes are hereby adopted and authorized to be prosecuted under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with the plans in the respective reports hereinafter designated: Provided, That penstocks or other similar facilities adapted to possible future use in the development of hydroelectric power shall be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers and of the Federal Power Commission.
>
> "MERRIMACK RIVER BASIN
>
> "The general comprehensive plan for flood control and other purposes, as approved by the Chief of Engineers pursuant to preliminary examinations and surveys authorized by the Act of June 22, 1936, is approved and the project for flood control in the Merrimack River Basin, as authorized by the Flood Control Act approved June 22, 1936, is modified to provide, in addition to the construction of a system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers."

The Bennington Reservoir was not included in the original comprehensive plan of reservoirs and related flood control works, but as a result of extensive investigations by and in accordance with the recommendations of the Board of Engineers for Rivers and Harbors, was approved by the Chief of Engineers in the 10th Indorsement dated 10 December 1942 on letter from the Chief of Engineers to the Resident Member, Board of Engineers for Rivers and Harbors, dated 6 December 1941, subject: "Reservoir Plans for the Contoocook Basin, New Hampshire," File No. 7402 (Merrimack River-Hopkinton-Everett Res.) 41, and was substituted together with Beards Brook Reservoir for the Hopkinton-Everett Reservoir.

The Bennington Dam is located on the Contoocook River 1/2 mile south of the village of Bennington, N.H. The dam consists of a gravity ogee type concrete overflow spillway section with centrally located outlets and a concrete non-overflow section adjoining each side of the spillway. The non-overflow sections then extend into rolled earth embankment sections which form the major portion of the dam. The initial dam has a total crest length of 3,520 ft. and will be founded on an impervious glacial till, except under a portion of the westerly abutment. It is proposed to construct the reservoir initially for flood control only and

provisions have been made in the design of the initial stage to enable raising the embankment 6 ft. and the spillway 7 ft., in the future, to provide for conservation storage in addition to flood control storage.

Financial local cooperation is not required for this work.

2. Investigations.- a. Scope of Investigations and Studies.- Investigations and studies have been made of all factors affecting the construction of the Bennington Reservoir. Data have been compiled, studied and analyzed on climatical, hydrological, and geological conditions, flood heights, frequencies and losses, power possibilities, economics of construction, and benefits to be derived from the construction of the project. Detailed descriptions of the investigations and analytical studies are contained in other sections of this report.

b. Previous Investigation of Contoocook River Basin.- A comprehensive system of flood control reservoirs and related flood control works in the Merrimack River Basin was authorized by the Flood Control Act of 28 June 1938 (Public No. 761 - 75th Congress 3rd Session) and funds were appropriated for the work. This authorization for a comprehensive system was based on a report and recommendations made by The District Engineer which were submitted to Congress and published as House Document No. 689, 75th Congress, 3rd Session. In the preparation of this report preliminary studies of the Bennington site were made and the site listed as a possible location for a reservoir, although due to local opposition at that time, construction was not recommended. For control of the Contoocook River this report placed emphasis on the Riverhill site, in the westerly part of Concord. Due to local opposition that developed, this site was abandoned and projects for the construction of the West Peterboro, Mountain Brook, and Hopkinton-Everett Reservoirs substituted.

Various reports were submitted on the above three reservoirs, including a definite project report on the Hopkinton-Everett project which was approved by the Chief of Engineers on 12 March 1940 File (E.D. 7402) (Merrimack River, Hopkinton-Everett Reservoir)-8, subject to minor modifications. During the early period of planning on this project the matter was referred to the Federal Power Commission for comment and recommendation. The Commission carried on extensive investigations of the possible overall development of the Contoocook River Basin, for which investigations the District Engineer obtained and furnished the basic data. The report and recommendations of the Commission were presented at the time the hearings were being held with reference to the request by the War Department for approval by the State of New Hampshire of the acquisition of land for the Hopkinton-Everett project. The Commission's report included proposals for the construction of a series of reservoirs including one at Bennington as an alternate for the Hopkinton-Everett project.

There then followed a series of studies made by the War Department and the Federal Power Commission, but since no agreement as to the best means of development could be reached the matter was referred to the Board of Engineers for Rivers and Harbors for recommendation, by letter from the Chief of Engineers dated 6 December 1941 subject: Reservoir Plans for the Contoocook Basin, New Hampshire, File 7402 (Merrimack River-Hopkinton-Everett Res.)-41. The recommendations of the Board of Engineers are contained in the 9th Indorsement to this letter and are briefly that the Bennington and Beards Brook Reservoirs be constructed in place of Hopkinton-Everett.

c. Investigations of Bennington Reservoir Site.- The reservoir capacity used in connection with all investigations of the Bennington site made prior to 1944 was based on data obtained from United States Geological Survey topographic maps.

Early in 1944 a topographic survey of the reservoir area showing five-foot contours, was made by aerial photographic methods. From the data thus obtained capacities were again computed, with results showing volumes considerable in excess of those previously used. A tabulated comparison of certain reservoir capacities used is given below:

Pool Elevation	Capacity in Acre Feet	
	From U.S.G.S. Topo. Maps Used in Prelim. Reports	From Aerial Survey Used in Definite Project Report.
700	30,000	41,000
705	44,000	60,000
710	60,000	80,000
712	66,000	90,000

The total capacity of 60,000 acre-feet given in the table on page 12 of the 9th Indorsement referred to in subparagraph b. above contemplated a pool elevation of 710. In accordance with the instructions from the Chief of Engineers contained in the succeeding 10th Indorsement this capacity is being provided in the initial development, but due to more complete capacity data, with pool elevation of 705.

The estimated cost of the project with the spillway crest at elevation 705 is slightly less than the cost estimated by the Board of Engineers.

Economic studies of the possibility of utilizing part of the storage at Bennington for conservation purposes have been made. These indicate that such storage can be economically provided.

Because of the heterogeneous foundation conditions and depth to bedrock existing throughout the entire area of the proposed dam site, numerous locations and arrangements of structures have been studied. Considerable foundation exploration work has been accomplished within the reach extending from the existing Monadnock Power Dam to approximately 1200 feet upstream from the Powder Mill Dam. The general plan of the dam proposed in this report is selected as the most feasible and economical layout with both the initial and ultimate developments in view.

d. Status of Approval by State of New Hampshire.- By letter dated 15 February 1945 to the Honorable Charles M. Dale, Governor of the State of New Hampshire, approval by the State of the acquisition of land for the Bennington Reservoir was requested. At the present time no action has been taken on this request.

3. Local Cooperation.- Local financial cooperation is not required as Section 2 of the Flood Control Act approved 28 June 1938 (Public No. 761 - 75th Congress, 3rd Session) applies to this project.

4. Definite Project Plan.- a. General.- The Bennington Dam is one of several reservoirs comprising the comprehensive plan for flood control reservoirs and related flood control work for the Merrimack River Basin. As noted in paragraph 1, the Bennington Reservoir was not included in the original comprehensive plan of reservoirs, but together with the Beards Brook Reservoir was substituted for the Hopkinton-Everett Reservoir for control of the head waters of the Contoocook River.

b. Location.- The proposed reservoir is located on the Contoocook River, N.H., in the Towns of Bennington, Hancock, Greenfield, and Peterboro, in the southwestern part of the State of New Hampshire. The dam is situated approximately one-half mile upstream (south) of the Village of Bennington, and just below a small storage reservoir of the Monadnock Paper Company, and above a pond formed by an intake dam owned by the same company. The storage reservoir of the Monadnock Paper Company just mentioned is operated for stream regulation only, the intake facilities for power and process water purposes are located at the intake dam downstream from the site of the proposed new dam.

c. Description of Reservoir Area.- A large part of the land within the reservoir basin is wooded and covered with a second growth composed mainly of white pine, hemlock, white and red maple, grey and silver birch, and white and red oak. Scattered about are clumps of white pine, hemlock and spruce of sufficient size to be merchantable. However, it is doubtful if there is sufficient amount of mature timber on any one unit to make commercial operations profitable. It is very noticeable that within the thin stands of birch and the like there is

a good undergrowth of young white pine. These trees appear thrifty and are making good growth.

The largest area of cleared land is situated along Ferguson Brook in the town of Hancock. Near the point where it enters the Contoocook River, this section is occupied by three good agricultural units. The large, almost level fields in this valley, with a soil of the Merrimack series, lend themselves to machine operation. Two of these units are operated as dairy units. Across the Contoocook River in the town of Greenfield, there is a poulty farm which has housing capacity for about one thousand layers. These four properties make up the major part of the agricultural operations now being conducted within the areas under consideration.

At the point where U.S. Route Number 202 crosses Ferguson Brook, there is a small saw mill. Ferguson Brook is the source of the power for this mill and the dam is located on the westerly side of Route Number 202 and just south of the mill. These facilities will have to be eliminated as they will be within the reservoir area.

In the ultimate development with the spillway crest at elevation 712, an additional area of approximately 700 acres will be required which will affect a number of properties situated at the upper, or Peterboro, end of the proposed reservoir area. Included in this area are thirteen (13) residential properties consisting of moderately priced homes, garages and out-buildings. These improvements are on average size lots which, as a rule, have well kept lawns with trees and shrubs well established. There are two oil companies in this same locality which are located on lots adjacent to the river, possessing the usual tanks, garages and out-buildings. There is also a new brick building in the area which is being used as a freeze-locker establishment.

d. Design Flood.- (1) Reservoir Design Flood.- Two large floods of record, March 1936 and September 1938, have occurred on the Contoocook River at the reservoir site. The largest of these floods, September 1938, had a peak discharge of 15,400 c.f.s., and a 6 day volume of 82,000 acre-feet. The second largest flood, the two peak flood of March 1936, had a maximum peak of 13,600 c.f.s. and a 13 day volume of 130,000 acre-feet. These floods were used to determine the storage capacity under the assumed conditions of reservoir operation.

(2) Spillway Design Flood.- The storm used for spillway design flood comprises the maximum possible rainfall over the 186 square miles drainage area or 17.2 inches of rainfall in 18 hours, with 13.9 inches occurring in 6 hours. Run-off was computed assuming an infiltration rate to be .05 inches per hour, and a base flow of 5 c.f.s. per square mile. By means of a synthetic unit hydrograph (see Plate I-12), the resulting spillway design flood was found to have a peak inflow of 77,600 c.f.s., and a

volume of 162,800 acre feet. The resulting spillway outflow was determined to be 45,900 c.f.s. at a reservoir stage of 716.8 feet or 11.8 feet above initial spillway crest.

e. Project Description.- As shown on Plates IV-1 to IV-5 accompanying Appendix IV, the impounding structure consists of a gravity ogee type concrete overflow spillway section with centrally located outlets, a concrete non-overflow section adjoining each side of the spillway and extending into the rolled earth embankment sections which form the major portion of the dam. The proposed initial dam has a total crest length of 3,520 feet and is founded on impervious glacial till, except a portion of the westerly embankment. It is proposed to construct the reservoir initially for flood control only, and provisions have been made in the design of the initial stage for raising the embankment 6 feet and the spillway 7 feet, in the future, to provide for conservation storage in addition to flood control storage. The reservoir, which controls a gross drainage area of 186 square miles and net drainage area of 128 square miles, has an area of 3,885 acres at the elevation of the spillway crest of the initial installation and an area of 4,550 acres at the elevation of the spillway crest of the ultimate installation. In the initial installation, the reservoir has a capacity of 60,000 acre feet which is equivalent to 6.0 inches of run-off in the gross drainage area, and in the ultimate installation, a capacity of 50,000 acre feet of flood control storage and 40,000 acre feet of conservation storage, making a total of 90,000 acre feet, which is equivalent to 9.0 inches of run-off on the gross drainage area.

f. Estimated Cost.- A detailed estimate of cost for the initial construction is given in paragraph 14. A summary of the estimated cost is as follows:

Reservoir Costs	$1,482,000
Reservoir Clearing	4,000
Construction Costs	2,514,000
Total Estimated Cost . . .	$4,000,000

g. Method of Operation.- The Bennington Reservoir will be operated primarily to regulate flood discharges of the Contoocook River so as to provide maximum benefits at downstream damage centers and to reduce flows in the river just below the dam to amounts equal to or less than channel capacity. During periods of normal flow the reservoir will be operated so as to provide the same storage and to discharge water to the mills in Bennington at rates equivalent to those that existed prior to the construction of the dam.

h. Alternative Location.- A number of engineering studies have been made for various locations within the vicinity of the proposed

dam site and considerable drill hole and seismic information has been obtained.

A study of the "Record of Foundation Exploration Plans," Plates II-4 to II-8 inclusive, will indicate the heterogeneous foundation conditions which limited the location of the dam to its proposed site without encroaching on the Village of Bennington or on the more pervious foundation that exists upstream. This site utilizes the underlying impervious glacial till to the fullest advantage, as rock within the site is at a considerable depth below the ground surface as indicated and is impractical to reach for a foundation for masonry structures or for the impervious core. Considerable investigation and drilling work was required in order to select a suitable location for the spillway and related masonry structures. The spillway, as proposed, is located on the highest till obtainable which is continuous to the underlying rock.

The site also takes advantage of the topography in that the high ground on either side of the valley in the vicinity of the selected centerline reduces the volume in the earth fill embankment to a minimum.

i. Ultimate Development.- In view of the favorable results of an analysis of this project for future conservation in addition to flood control storage, provision has been made in the initial stage design for an ultimate addition of 7 feet to the spillway and 6 feet to the embankment. Ultimate conservation is more fully discussed in Appendix V.

j. Construction Procedure.- During the first season of construction, it is proposed to construct a portion of the west embankment to full height, the east embankment to El. 679.5, and the concrete structures to the elevations as indicated on Plate IV-6, allowing the river to flow in its natural stream bed. During the second season the work proposed consists of constructing the cofferdams and diversion channel for diverting the water through the outlets, as shown on the plate noted above, and completing the embankment and masonry structures. Highway traffic can be maintained during the various stages of construction by re-routing until such time as the proposed raising of roads and relocation, discussed elsewhere, are completed.

5. Structures and Improvements.- a. Spillway Structures.- The spillway structure is a concrete gravity wide-base ogee type section with a crest length of 300 feet at elevation 705 and maximum height of 49 feet, with provisions made in the initial design for an ultimate addition of 7 feet to accommodate an additional 30,000 acre feet of storage. Six 4'-0" x 6'-0" conduits, invert elevation 667 with hydraulically operated sluice gates are centrally located in the spillway for control of discharge. One emergency gate, operated from the

exterior operating platform, is provided in the event of failure of the hydraulically operated gates. Access is gained to the gate chambers from the Equipment House which is located on the downstream berm on the east side of the stilling basin, by means of a passageway incorporated into the stilling basin gravity wall, then through the spillway section to the various gate chambers. The passageway then continues to an adit on the west side of the spillway providing an emergency exit and convenient method of access from one side of the dam to the other. The spillway is founded entirely on on an impervious glacial till deposit that extends in depth to the underlying rock. The spillway section was designed for the ultimate development and proportioned to obtain a minimum factor of safety against sliding of 1.5 with the maximum foundation pressures ranging from 4.3 tons per square foot with an empty reservoir to 2.7 tons per square foot with a full reservoir. A detailed analysis of design is discussed in Appendix IV. In the initial development with the spillway crest at elevation 705, a surcharge of 11.8 feet will be required to pass the spillway design flood discharge of 45,900 cubic feet per second. Similarly, in the ultimate development with the spillway crest at elevation 712, a surcharge of 11.1 feet will be required to pass the spillway design flood discharge of 42,300 cubic feet per second. Under conditions of the spillway design flood there will be a freeboard of 7.2 feet in the initial development and 6.9 feet in the ultimate development. The maximum flood of record occurred in 1938 with a peak discharge of 15,400 cubic feet per second.

b. Embankment.- The embankment section of the dam is an earth fill section with a dumped rock blanket on the upstream face and downstream toe, and a facing on the downstream slope of raked gravel, as indicated in the section on Plate IV-2. The impervious core cut-off extends into the glacial till foundation with the exception of that portion of the embankment between Sta. 28 + 15 and 36 + 90 on the westerly side where the overlying pervious materials are too thick for the economical construction of a core cut-off. An inspection trench has been provided and provision is made in the initial development for the construction of an impervious blanket tied into the core on the upstream side of the dam. The top width of the dam at elevation 724 is 25 feet with an average slope of 1 on 2-1/2 on the upstream side and 1 on 2-1/4 on the downstream side. The top width and side slopes

of the ultimate embankment will be similar to that of the initial construction.

c. Non-Overflow Sections.- The non-overflow sections are concrete gravity-type structures and extend from the spillway into the earth embankment on both sides and are founded on the underlying deposit of impervious glacial till. A considerable saving is made in the use of these non-overflow sections as compared with the high retaining walls which would otherwise be required along each side of the spillway channel. A number of comparative estimates have been prepared of various types of wall design and non-overflow sections. The design proposed is considered more satisfactory and has proved less expensive.

d. Stilling Basin Walls.- The stilling basin walls are concrete gravity-type sections with one foot of porous concrete on the bottom of the section placed there to relieve the uplift pressure and also to drain the fill retained by the walls.

A passageway is incorporated into the east wall which provides access from the Equipment House to the operating chambers in the spillway and thence to the adit on the west side of the spillway.

The Equipment House which is located on the downstream berm on the east abutment is founded on the stilling basin wall and piers with spread footings.

e. Stilling Basin.- The stilling basin is a reinforced concrete mat with reinforced concrete baffles and end sill. The bottom foot of the mat is composed of porous concrete which reduces the uplift by allowing free drainage to the perforated tile collector pipes located under the mat. These collector pipes in turn drain into the wells provided in the retaining walls which have outlets into the stilling basin.

f. Drainage Wells.- A series of drainage wells, as shown on Plate IV-1, have been incorporated into the design of the dam on the downstream side of the west abutment. These wells were introduced to reduce the hydrostatic head in the

foundation at the downstream toe of the embankment during flood stage periods. For the ultimate development, provisions also have been made for an impervious blanket on the upstream side of the westerly section of the embankment. For the initial development, an impervious blanket is provided under the pervious and random fill sections on the upstream side of the core in the westerly section of the embankment and is carried 10 feet beyond the toe of the embankment so that the ultimate blanket can be added without disturbance to the embankment.

6. Foundations.- a. Geological Setting.- The Contoocook River stream channel of the pre-glacial period was a bedrock valley of porphyritic granite. The topography of the region was modified by glaciation forming the present stream valley on the east wall of the original valley. During the recession of the glacier, the northward flow of water was impeded forming a glacial lake in which large quantities of sand and silt were deposited.

b. Occurrence and Extent of Foundation Materials.- Extensive investigations of foundation condition at the site have been made by means of test pits, borings and seismic explorations, and it has been found that in the immediate project area surface deposits of variable gravelly and silty sands occur to a depth of 10 to 15 feet. Under these surface deposits the soil body in the east bank is a compact glacial till, continuous to bedrock. This till extends under the river and tapers out in the west bank where the till is underlain by variable sands and silt. Except for a small area where till exists as a thin layer near the ground surface the west bank is composed entirely of sand and silt. Bedrock of porphyritic granite, with a fractured and weathered capping, varies in depth below existing ground surface from approximately 50 feet in the east bank to more than 150 feet in the west bank.

c. Suitability of Foundation for Required Structures.- All foundation materials have high shear strength and good bearing capacity. The till is relatively impervious and the other variable deposits range in relative permeability from pervious to semi-impervious materials. A summary of the foundation investigations and analysis is presented in Appendix II of this report.

7. Conservation Storage.- a. Studies.- Studies of the feasibility of providing storage at the Bennington Reservoir for flood control and for stream regulation indicate that such development would be economically justified due to the comparatively cheap

storage obtainable. The elevation of the spillway crest can be raised seven feet from elevation 705 as selected for the flood control structure, to elevation 712. At this latter elevation, which is the maximum feasible due to damages which would be caused to the town of Peterboro, N. H., by higher backwater, a total storage capacity of 90,000 A.F. would be created, of which 50,000 A.F. would be reserved for flood control storage with 40,000 acre-feet remaining for stream regulation. The cost of storage for the recommended project as applied to the various types of storage proposed is summarized as follows:

	Cost
Flood Control Project with Spillway Crest elevation 705, (capacity 60,000 acre-feet) including allowance of $114,000 for modifying initial flood control structure to permit future raising	$4,000,000
Future raising of structures and Reservoir with Spillway Crest elevation 712, capacity 90,000 acre-feet	$1,531,000

b. Estimate of Costs and Benefits.- Second stage construction, besides raising the dam and spillway elevations, would involve the acquisition of 800 acres of additional Reservoir Area and the raising of 1.5 miles and the relocating of 3 miles of highways. The estimated costs of the project applicable to the various storage uses and construction stages are tabulated on Plate V-1.

Studies in Appendix V show that the existing power installations on the Contoocook and Merrimack Rivers, representing 380 feet of developed head with a total installed capacity of 70,500 KW would benefit from the increased low-water flow due to operation of conservation storage at Bennington to the extent of 9.8 million KWH annually. With the reservoir operated to produce a minimum regulated discharge of not less than 100 c.f.s. the mean low-water flow at Manchester would be increased by about 150 c.f.s., raising the prime peaking capacity of the existing installations on the Merrimack River by about 2600 KW. In addition, the small plants on the Contoocook River will benefit, and unsanitary conditions in the rivers will be mitigated to some extent. The studies of the benefits that would be derived from a combined conservation and flood control reservoir as shown on Table A, Appendix V, fully justify present planning for future second stage construction.

8. Relocations.- a. Railroads.- A branch line of the Boston and Maine Railroad is now located within the reservoir basin. This line originates at Nashua, New Hampshire, runs northwesterly through the villages of Greenfield, Elmwood and Bennington, and extends as far north as Hillsboro, New Hampshire. It is proposed to eliminate this line from Greenfield to Bennington and rehabilitate the former line between West Henniker and Hillsboro so that Bennington can then be serviced from the Concord branch. Studies of the possible relocation of the existing line at the dam site have been made by both the Boston and Maine Railroad and the U. S. Engineer Department and such relocation has been found to be impractical and costly.

b. Highways and Roads.- There is one main highway and some second and third class roads within the limits of the proposed reservoir that will require relocation or raising as shown on Plate VI-1.

U. S. Highway Route No. 202, which is a first-class highway, follows along the westerly side of the river connecting Peterboro and Bennington, and is subject to inundation for a total length of approximately 3 miles. It is proposed to relocate the affected portions of the road on high ground to the west of their present sites and at an elevation high enough to meet the requirements of the ultimate development.

The second-class surfaced highway on the easterly side of the river passes through the proposed dam site, and it is proposed to relocate this section on high ground easterly of its present site. The access road to the dam will then tie into this relocated road and provide an entrance to the dam site from the larger villages in the vicinity. Other sections of this road that will be inundated during flood stages in the initial development will be raised to a minimum elevation of 708.0, and to elevation 715.0 at a time when the ultimate development is constructed.

In the initial stage, in order to provide a connection between Greenfield and Hancock during periods of high water other than extreme high flood stages, it is proposed to raise the road within the limits of the reservoir, to elevation 695.0, and to raise the bridge to obtain clearance to pass corresponding flood flows. In the ultimate stage, it is proposed to relocate this road on the higher ground just downstream from its present location and construct a new bridge.

c. Other Facilities.- There are no cemeteries or major pipe lines within the reservoir area. There is a trunk telephone line in the area that will require relocation, and other communications lines and power lines that serve only the immediate vicinity which, in general, will no longer be required after the reservoir is constructed.

d. Method of Accomplishing Relocations.- It is proposed to accomplish relocation of utilities by contract with the respective owners.

9. Availability of Construction Materials.- a. A large part of the required construction materials are available from structure excavation. A very small amount of the structure excavation is suitable for impervious fill in the central core of the dam; a small amount is suitable for the free draining sections of the dam; but the major portion of the excavation is suitable only for compacted random fill in the embankment or for semi-compacted fill in other portions of the project. Suitable borrow material for impervious fill is located approximately 2,000 feet northeast of the spillway area. This borrow area has an overlying deposit of variable sand suitable only for random fill. An adequate quantity of free draining gravel and sand for pervious fill is available in an esker located southeast of the dam and extending approximately 1/2 mile from the approach channel. Concrete aggregates and processed materials for filters, drains, and riprap backing are available from this area. Rock for slopeprotection is available as oversized material in required structure and borrow excavations, from accumulation of surface boulders in the general vicinity of the dam and by quarrying at two locations at distances from the dam site of 3 and 6 miles respectively.

b. A summary of materials available from excavation with indicated disposition, and materials required for construction with indicated source, are presented in the tabulation given in Paragraph c of Appendix II.

10. Construction Time Required and Schedule of Operations.- a. Required Construction Time.- The construction of the initial stage of the dam is scheduled over a two-year period based upon the execution of all construction by contract as indicated graphically on Plate IV-6.

b. Schedule of Operations.- The schedule of construction operations based upon completion of the dam in two years is as follows:

First Season

Construct embankment between Station 20 + 00 and Station 36 + 90, to elevation 724.

Construct east embankment to elevation 679.5.

Excavate area for spillway, stilling basin, non-overflow sections and stilling basin walls.

Construct spillway to elevation 679.5, non-overflow sections to elevation 679.5, stilling basin and stilling basin walls complete.

Excavate approach channel, except a portion at river bank to serve as a cofferdam.

Excavate spillway discharge channel.

Second Season

Complete spillway and non-overflow sections.

Construct upstream and downstream cofferdams.

Excavate diversion channel.

Dewater and construct section of embankment between Station 11 + 38 and Station 20 + 00, to elevation 724, and complete each embankment between Station 1 + 70 and Station 8 + 38.

Construct downstream terrace and stone dike.

Construct Equipment House.

Remove upstream cofferdam, grade area between approach channel and toe of dam, remove closure cofferdam at entrance to approach channel.

Relocation and raising of roads and other facilities in the reservoir area would be accomplished largely during the second construction season.

c. Funds Required by Fiscal Years.- The funds required during each fiscal year of the two-year construction period, for the accomplishment of the project by contract, including land acquisition and relocations, construction operations, and engineering, contingencies and overhead, are estimated to be as follows:

First Year	$1,900,000
Second Year	2,100,000
Total	$4,000,000

d. Preparation of Plans and Specifications.- It is

estimated that a period of four months will be required for the preparation of contract plans and specifications at a total estimated cost of $50,000.

e. Employment Analysis.- In comparison with similar projects constructed in the Boston District, it is estimated that the project reported herein will create the following number of man-hours of labor:

(1) At-Site Labor:

Skilled Labor	350,000	man hours
Unskilled Labor	1,400,000	" "
Other	100,000	" "
TOTAL	1,850,000	man hours

(2) Off-Site Labor:

TOTAL	300,000	man hours

11. Clearance with other Agencies.- In accordance with Circular Letter No. 2652, File No. 800.12 (Cooperative Procedure), dated 1 January 1944, subject: "Quadripartite Agreement," and Circular Letter No. 2306, dated 1 August 1944, subject: "Fish and Wild Life Service Cooperation," information and pertinent data on the construction of the proposed Bennington Reservoir has been imparted to the Soil Conservation Service of the Department of Agriculture, Federal Power Commission, and the Fish and Wild Life Service of the Department of the Interior.

Local representatives of the Fish and Wild Life Service have been consulted and the details of the project have been discussed with them. Plans and other information have been furnished at their request, and it is understood that a complete investigation and study is being made of the project by this Service.

Several conferences have been held in the Boston District Office with a representative of the Federal Power Commission. The initial and ultimate construction and relationship of the proposed Bennington Reservoir as part of the comprehensive plan for flood control of the Merrimack River Basin has been discussed in detail with representatives of this Department. Folios prepared for the Board of Consultants conferences containing complete plans and analysis of the dam were furnished the Federal Power Commission for its use. No formal reply has been received by this office in regard to any investigation the Commission might be undertaking at this time.

In accordance with the provisions of the Flood Control Act,

approved 22 December 1944 (Public Law 534 - 78th Congress - 2nd Session) and as this project in its entirety lies east of the ninety-seventh meridian, the Department of the Interior has not been consulted with reference to matters of land reclamation.

The Soil Conservation Service, Department of Agriculture, has corresponded with this office and forwarded its "Report - Soil Erosion Conditions in New Hampshire," for review and wishes to be informed of any hearings which may be held. No further action by the Department has been indicated to date.

12. Operation and Maintenance.- The operation and maintenance of the dam will be a Federal function with the principal operational duties consisting of gate operation. As described in detail in Appendix I, the maintenance of a pool between elevation 667 and 678 to provide the same storage in the future as is presently furnished by the pond at the Powder Mill Dam will require daily operation of the gates. Likewise, any increase in pool elevation above elevation 678 will require frequent gate operation.

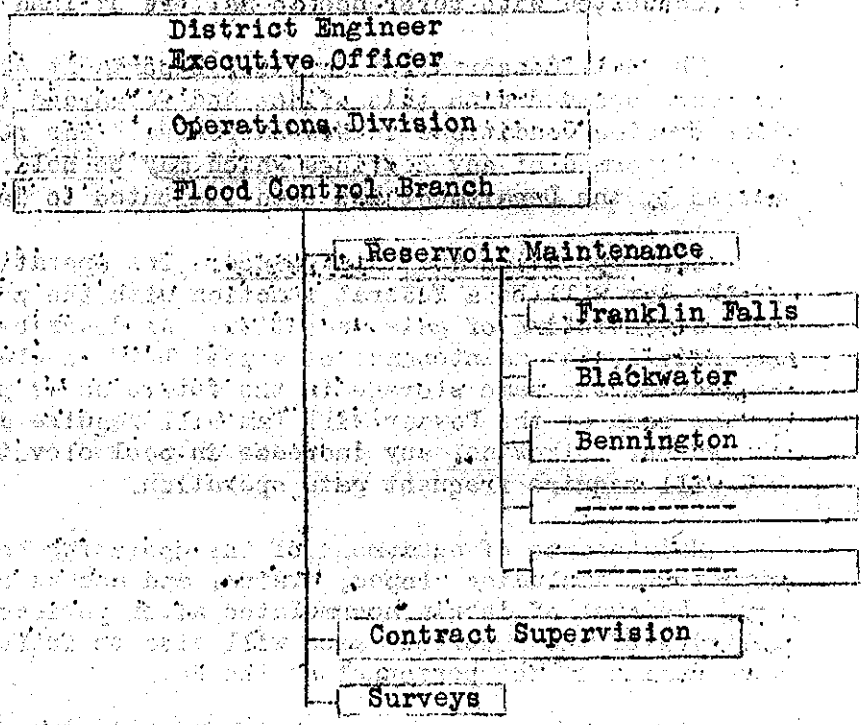
Maintenance of equipment of the operating house, and of the dam structure, including slopes, drains, and access road, will be necessary. Removal of debris accumulated after periods of high water and policing of the reservoir area will also be included in the maintenance duties of the personnel at the dam.

The permanent organization at the site will be made up of two classified personnel and one laborer as follows:

Bennington Dam Reservoir Operations and Maintenance
Superintendent, P-2, $2600
Damtender, CPC-6, $1800
Laborer - $1260

Quarters will be provided at the dam for the superintendent. When additional laborers are required for such special jobs as removal of large amounts of debris, or in case of emergency, they will be hired locally and employed only for the period required.

The operation and maintenance force will be under the supervision of the Flood Control Branch of the Operations Division of the Boston District Office. The organizational set-up would be:



The estimated annual cost of operation and maintenance is as follows:

Item of Work	Estimated Cost
Services of Operators and Helpers	$ 7,200.00
Power, lubricants, and fuel for operation of gates	800.00
Heating operating structure	400.00
Clearing and removal of debris	3,250.00
Landscaping	500.00
Maintenance of access road, surface drains, and unprotected slopes	700.00
Repairs and painting	800.00
Miscellaneous	300.00
District Office overhead	1,050.00
	$ 15,000.00

13. Malaria Control.- In accordance with instructions contained in Circular Letter No. 3606, dated 9 March 1945, concerning "Malaria Control at River and Harbor and Flood Control Reservoirs", this office has requested the advice and recommendations of the U. S. Public Health Service with respect to the need and requirements for malaria control at the Bennington Reservoir. Plans and pertinent data relative to the reservoir have been forwarded to the Public Health Service, but to date no reply has been received. When received, it is proposed to include the review of the Public Health Service in the project report as a supplementary appendix.

14. Cost Estimates.- a. Total Cost.- The estimated costs, including engineering, contingencies and overhead, for the principal elements of the project are as follows:

Reservoir Costs and Relocations	$1,482,000
Reservoir Clearing	4,000
Construction Costs	2,514,000
Total Estimated Cost	$4,000,000

b. Unit Costs:- The above estimated costs of principal elements are based upon the estimated quantities and unit prices as detailed in the following table. The estimated value of lands and damages are based on an appraisal made by the New England Division Office, Boston, Mass., and are believed to represent a reasonable evaluation. The reservoir area, at spillway lip elevation 705, contains 3885 acres, of which 25% is scrub growth, 53% is wooded, 12% is tillable, and 10% is inundated.

DETAILED ESTIMATE OF COSTS

I. RESERVOIR COSTS

	Quantity	Unit	Unit Price	Total Cost
Land and improvements	--	--	Lump Sum	$ 240,500
Riparian and water rights	--	--	Lump Sum	20,000
Relocation of telephone and power lines and highways	--	--	Lump Sum	655,000
Relocation of railroad	--	--	Lump Sum	256,000
Sub-total - Reservoir Costs				$1,171,500
Contingencies (15%)				175,725
				$1,347,225
Government Expenses (10%?)				134,775
TOTAL RESERVOIR COSTS				$1,482,000

II. CONSTRUCTION COSTS

a. Earth Dam, Non-Overflow Section and Spillway

Removal of existing structures	--	--	Lump Sum	$ 2,000
Stream diversion and pumping	--	--	Lump Sum	40,000
Clearing and grubbing	90	Acre	300.00	27,000
Stripping	168,000	c.y.	.50	84,000
Excavation	486,000	c.y.	.40	194,400
Borrow - Impervious	165,000	c.y.	.55	90,750
Borrow - Pervious	350,000	c.y.	.50	175,000
Borrow - Random	110,000	c.y.	.50	55,000
Borrow - Rock	20,000	c.y.	4.00	80,000
Rolled fill - Impervious	148,000	c.y.	.15	22,200
Rolled fill - Pervious	200,000	c.y.	.12	24,000
Rolled fill - Random	195,000	c.y.	.12	23,400
Rolled fill - Semi-compacted	177,500	c.y.	.10	17,750
Structure backfill	18,500	c.y.	.60	11,100
Screened gravel backing	29,500	c.y.	2.00	59,000
Filter sand and gravel	48,000	c.y.	1.30	62,400
Gravel facing	8,500	c.y.	1.25	10,625
Dumped riprap	85,500	c.y.	.60	51,300
Derrick stone	5,000	c.y.	5.00	25,000
Rock surfacing	2,700	s.y.	0.30	810
Concrete - Spillway, Stilling Basin and Non-overflow Section	45,000	c.y.	13.50	607,500
Concrete-Stilling Basin Walls	7,000	c.y.	15.00	117,000
Reinforcing Steel	595,000	lb.	.06	35,700
Well System	--	--	Lump Sum	24,000
Equipment House and Operators Quarters	--	--	Lump Sum	25,000
Misc. Metals, Trash Bars, Emerg. Gates	--	--	Lump Sum	13,000
Gates and Hoists	--	--	Lump Sum	60,000

II. CONSTRUCTION COSTS (CONTINUED)

	Quantity	Unit	Unit Price	Total Cost
Lighting and Power system	--	--	Lump Sum	$ 15,000
Oil pressure system and Misc. equipment	--	--	Lump Sum	10,000
Miscellaneous items	--	--	Lump Sum	48,065
Sub-total - Construction Costs				$2,011,000
Eng'r Inspection, Overhead and Contingencies (25% ±)				503,000
TOTAL CONSTRUCTION COSTS				$2,514,000
III. CLEARING COSTS				
Reservoir Clearing				$ 3,000
Sub-total - Clearing Costs				$ 3,000
Government Expenses (25% ±)				1,000
TOTAL CLEARING COSTS				$ 4,000
IV. TOTAL ESTIMATED COST (I + II + III) =				$4,000,000

V. UNIT STORAGE COST

Cost per Acre Foot of Storage $\frac{4,000,000}{60,000} = \$66.67$

c. Carrying Charges.- The total annual carrying charge for the initial stage construction and ultimate increment of development of the reservoir, based upon interest on investment, on amortization of structures and equipment, and on operation and maintenance, is $177,018 for the initial stage development and $66,977 for the proposed ultimate increment of development. A break-down of these carrying charges is contained in the following tables:

Initial Stage

ANNUAL COSTS AND CARRYING CHARGES

I FEDERAL INVESTMENT

1. Total First Cost:		
a. Structures with 50 year life	$ 3,902,000	
b. Equipment with 25 year life	98,000	
c. Total	$ 4,000,000	
2. Interest During Construction: (3% for one-half construction period)		
a. On structures with 50 year life	$ 117,060	
b. On equipment with 25 year life	2,940	
3. Total Investment:		
a. Structures with 50 year life	$ 4,019,060	
b. Equipment with 25 year life	100,940	
c. Total Federal Investment	$ 4,120,000	

II ANNUAL FEDERAL CARRYING CHARGES

1. Interest on Investment @ 3%	$	123,600
2. Amortization:		
a. Structures with 50 year life (0.887%)..		35,649
b. Equipment with 25 year life (2.743%)...		2,769
3. Operation and Maintenance		15,000
4. Total Annual Federal Carrying Charges	$	177,018*

Construction Period - 2 years

* The annual charges as noted will be decreased by $4,565 giving a total annual carrying charge of $172,453, due to the provisions made in the initial stage for ultimate raising of the dam.

Ultimate Increment of Development

ANNUAL COSTS AND CARRYING CHARGES

I FEDERAL INVESTMENT

1.	Total First Cost:		
	a. Structures with 50 year life	1,526,890 114,000*	$ 1,640,890
	b. Equipment with 25 year life		4,110
	c. Total		$ 1,645,000
2.	Interest During Construction: (3% for one-half construction period)		
	a. On structures with 50 year life		$ 24,613
	b. On equipment with 25 year life		62
3.	Total Investment:		
	a. Structures with 50 year life		$ 1,665,503
	b. Equipment with 25 year life		4,172
	c. Total Federal Investment		$ 1,669,675

II ANNUAL FEDERAL CARRYING CHARGES

1.	Interest on Investment @ 3%		$ 50,090
2.	Amortization:		
	a. Structures with 50 year life (0.887%)		14,773
	b. Equipment with 25 year life (2.743%)		114
3.	Operation and Maintenance		2,000
4.	Total Annual Federal Carrying Charges		$ 66,977

Construction Period - 1 year

* Cost of modifying flood control structures to permit future raising.

15, Economic Study.- The Bennington Reservoir is proposed as part of the authorized comprehensive reservoir system for the Merrimack River Basin. A study has been made of the economics involving the construction of the dam as a flood control storage reservoir with provision for an ultimate addition to provide for conservation storage. As noted in the accompanying table "Summary of Benefits and Costs", the ratio of Annual Benefits to Annual Carrying Charges is 1.31 and therefore justifies the construction of a flood control reservoir with provision for an ultimate addition.

Economic studies have also been made for construction of the dam as a multiple-purpose reservoir, results of which are presented in Appendix V.

INITIAL STAGE

SUMMARY OF BENEFITS AND COSTS

1. Construction Costs:

a. Bennington Reservoir	$ 4,000,000
b. Other Flood Control Reservoirs (x)	14,224,000
c. Local Protection Projects (xx)	1,313,150
Total Construction costs	$19,537,150

2. Annual Carrying Charges:

a. Bennington Reservoir	$ 177,018
b. Other Flood Control Reservoirs (x)	661,518
c. Local Protection Projects (xx)	63,800
Total Annual Carrying Charges	902,336

3. Total Annual Benefits:
Based on comprehensive flood control program including reservoirs and local protection ..... $ 1,183,000

4. Ratio of Benefits to Carrying Charges:
Ratio of total Annual Benefits to Annual Carrying Charges ..... $ 1.31

(x) Includes completed reservoirs at Franklin Falls and Blackwater and proposed reservoirs at Beards Brook, Mountain Brook and West Peterboro.

	Cost		Annual Charges
Franklin Falls	$ 7,690,000	- Completed	$ 362,200
Blackwater	1,160,000	- Completed	59,000
Beards Brook	3,500,000	- Estimated	158,000
Mountain Brook	480,000	- Estimated	21,800
West Peterboro	1,394,000	- Estimated	60,518
	$14,224,000		$ 661,518

(xx) Includes completed local protection project at Lowell and proposed projects at North Andover and Lawrence, Mass., and Nashua, New Hampshire.

	Cost		Annual Charges
Lowell, Mass.	$ 443,500	- Completed	$ 25,500
North Andover, Mass.	323,400	- Estimated	13,880
Lawrence, Mass.	329,250	- Estimated	14,180
Nashua, New Hampshire	217,000	- Estimated	10,240
	$ 1,313,150		$ 63,800

16. Recommendations.- It is recommended that the construction of the Bennington Reservoir be authorized for flood control purposes with an initial spillway crest elevation of 705, and with provisions incorporated for future raising of the dam, to provide conservation storage with the spillway crest at elevation 712, all as described in this report and appendices.

HOMER B, PETTIT
Colonel, Corps of Engineers
District Engineer

SECTION D.

APPENDICES

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

GENERAL PLANS OF PROJECT

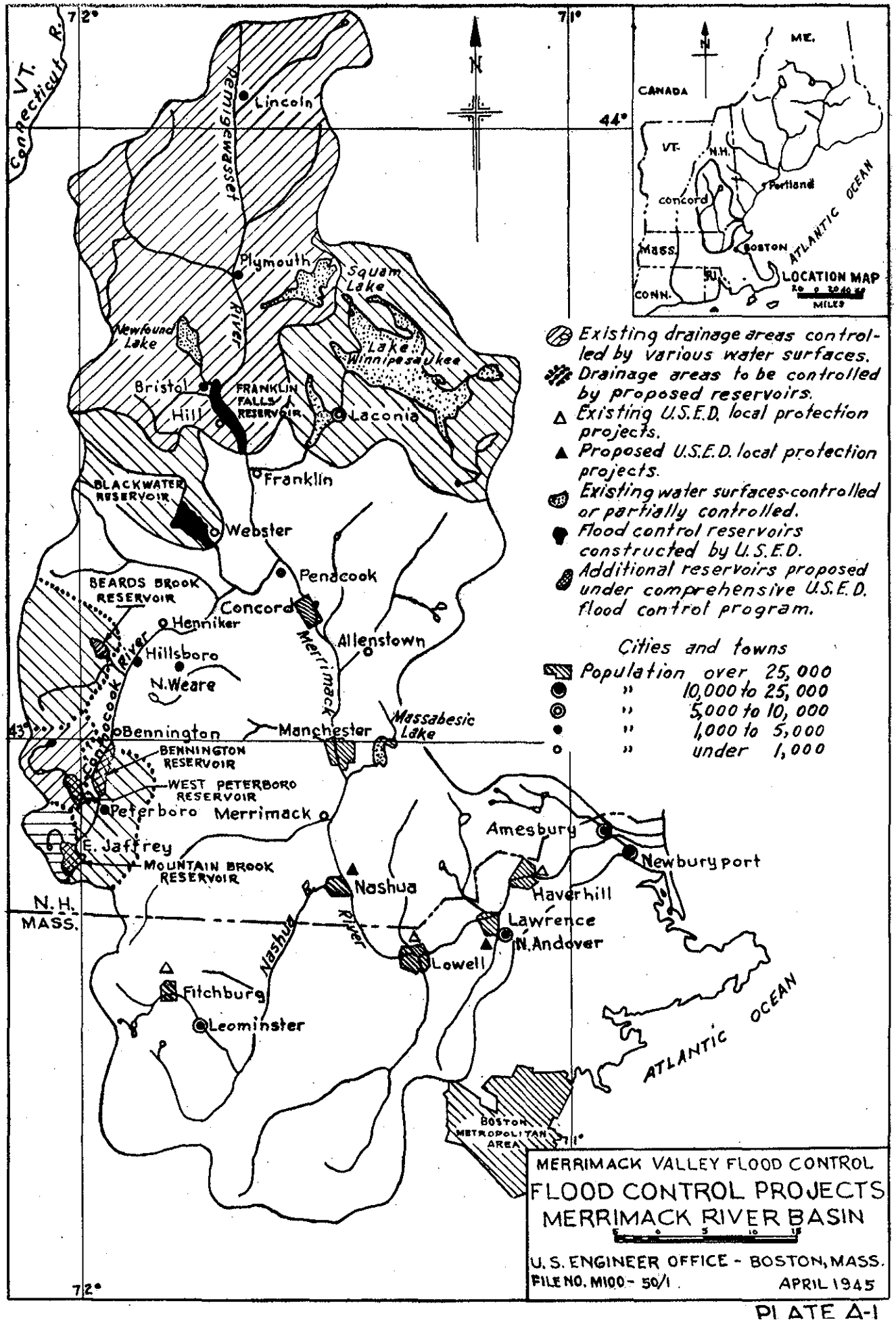
To accompany definite project report
Dated April 1945

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

GENERAL PLATES

Plate	Title
A-1	Flood Control Projects, Merrimack River Basin
A-2	River Profiles, Proposed Comprehensive Flood Protection System
A-3	Contoocook Valley, Location Map of Proposed Reservoirs and Drainage Basins
A-4	Reservoir Map
A-5	Aerial Photo of Bennington Dam Site
A-6	Plan of Dam and Vicinity



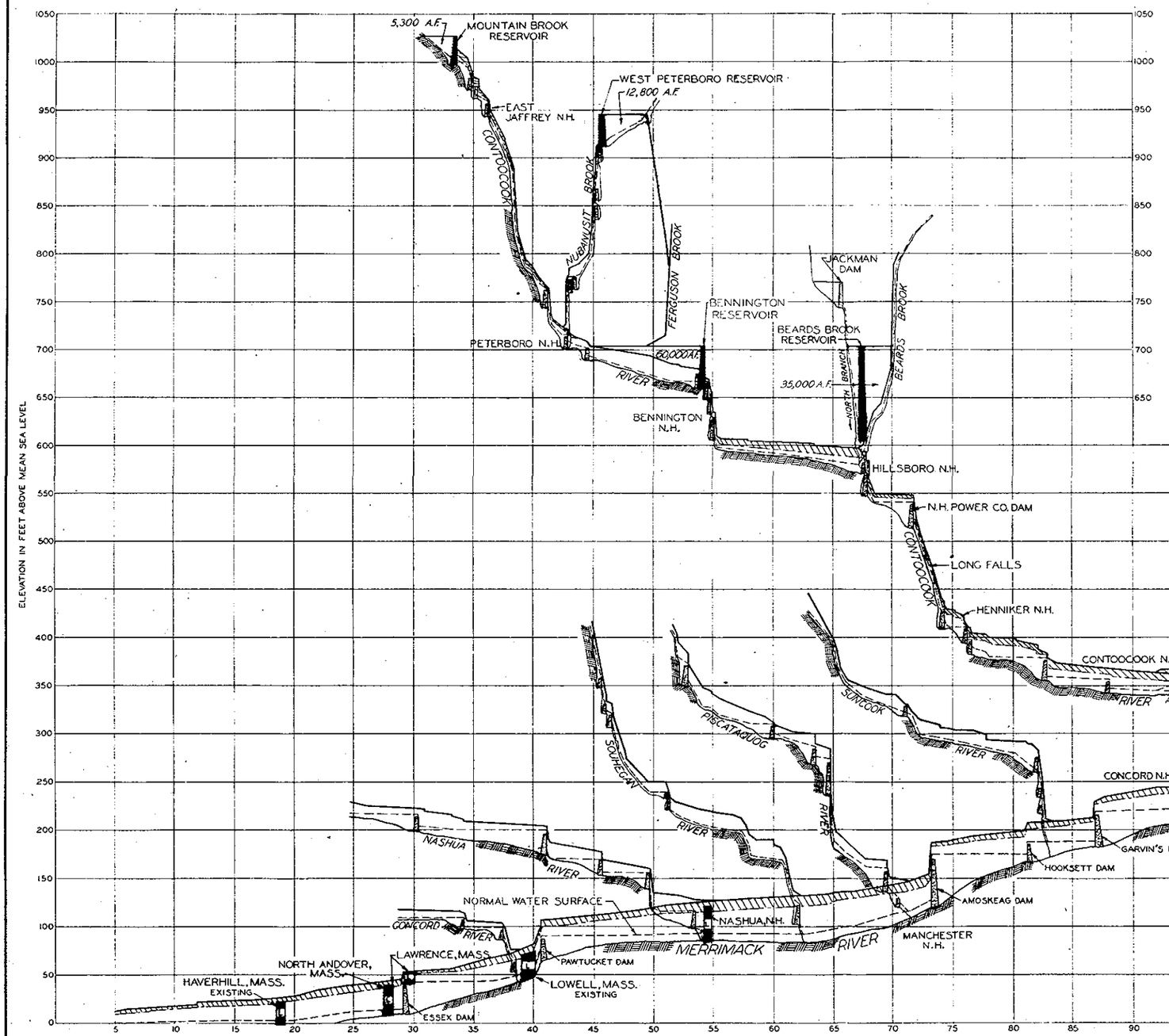
MERRIMACK VALLEY FLOOD CONTROL

FLOOD CONTROL PROJECTS MERRIMACK RIVER BASIN

U. S. ENGINEER OFFICE - BOSTON, MASS.

FILE NO. M100 - 50/1 APRIL 1945

PLATE A-1



LEGEND

- Existing and proposed Flood Control Reservoirs.
- Existing Utility Dams.
- Existing and proposed Local Protection.
- 1936 Flood Surface.
- Normal low water surface.
- 1936 Flood Reduced by Comprehensive Reservoir System.
- Stage Reduction by Comprehensive Reservoir System.
- Approximate river bed.

NOTE

Miles above Newburyport Light for tributaries, other than the Pemigewasset River, may be obtained by adding incremental mileage above point of confluence with the Merrimack River to the established mileage shown for point of confluence.

MERRIMACK VALLEY FLOOD CONTROL
RIVER PROFILES
SHOWING
PROPOSED COMPREHENSIVE FLOOD PROTECTION SYSTEM

U. S. ENGINEER OFFICE, BOSTON, MASS. FEB. 1935

APPROVAL RECOMMENDED
John L. Allen
CHIEF ENGINEERING DIVISION

APPROVED:
Irwin B. Pratt
BRIG. GEN. CORPS OF ENGINEERS
DISTRICT ENGINEER

SUBMITTED:
CHIEF DESIGN SECTION, ENGINEER

Edward F. Chase
ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M100-50/2

NO.	CHARACTER	DATE	BY
	REVISIONS		

PLATE A-2

BEARDS BROOK
DRAINAGE AREA
56 SQ. MILES

BEARDS BROOK
SPILLWAY CREST EL. 705

NORTH BRANCH
DRAINAGE AREA
64 SQ. MILES

BENNINGTON DAM
CREST ELEV. 705.

WEST PETERBORO
DRAINAGE AREA
44 SQ. MILES

Spillway

W. PETERBORO DAM
TOP EL. 967

BENNINGTON
TOTAL DRAINAGE AREA
186 SQ. MILES

MOUNTAIN BROOK
DRAINAGE AREA
14 SQ. MILES

MT. BROOK DAM
CREST EL. 1027

MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK VALLEY
LOCATION MAP OF
PROPOSED RESERVOIRS
& DRAINAGE BASINS

½ 0 1 2
SCALE IN MILES

U.S. ENGINEER OFFICE
BOSTON, MASS.

FILE NO. M100-50/3 PLATE A-3

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
RESERVOIR MAP
SCALE IN MILES 0 1/2 1 MILE
U.S. ENGINEER OFFICE, BOSTON, MASS
APRIL, 1945 FILE NO. M19-13/1

PLATE-A4

MERRIMACK VALLEY FLOOD CONTROL

AERIAL PHOTO OF

BENNINGTON DAM SITE

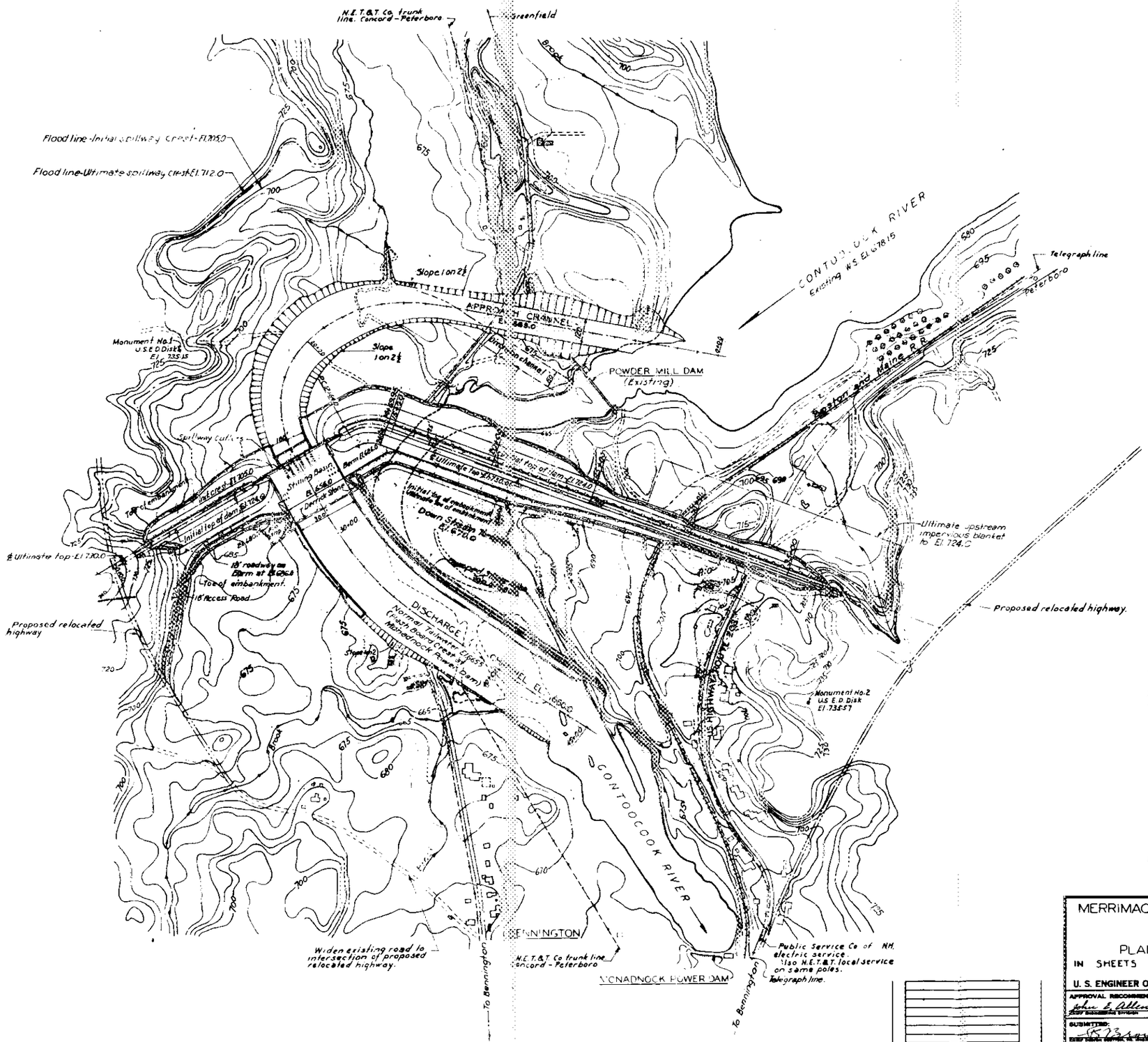
SCALE IN FEET

1000 0 1000

U.S. ENGINEER OFFICE BOSTON, MASS.

NOVEMBER 1944

PLATE A-5



LEGEND

- Existing building
- Existing building to be removed.
- Existing foundation.
- Existing hard surface road.
- Existing loose surface road.
- Proposed relocated highway.
- Denotes Initial Development
- Denotes Ultimate Development

Notes
Topography based on U.S.C.&G.S. datum of Mean Sea Level.
Topography traced from survey made by Fairchild Aerial Surveys, Feb. 1944.

200' 0 200' 400'
SCALE OF FEET

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
PLAN OF DAM AND VICINITY

IN SHEETS SHEET NO.

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: John E. Allen
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SUBMITTED:

NO.	CHARACTER	DATE	BY
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FILE NO. M19-13/3

PLATE A-6

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX I

HYDROLOGY

To accompany definite project report
Dated April 1945

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX I - HYDROLOGY

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APPENDIX I

HYDROLOGY - GENERAL DESCRIPTION OF PROJECT AND REGION

a. Scope.- The data contained in this appendix constitute a report on the hydrology and topographic features of the drainage basin of the Bennington Reservoir as affecting the spillway design flood and spillway requirements in accordance with the procedure outlined in the Engineer Bulletin R. & H. No. 9, 1938. In addition, the report contains the data and procedure used in determining the reservoir design flood.

b. General Discussion of Climate.- A general summary of the climate prevailing over the Contoocook River watershed can best be described by reference to page 5, Chapter 1 of Hydrometeorological Report No. 1 "Maximum Possible Precipitation Ompompanoosuc Basin," which states:

"The entire region is so situated geographically that it receives a maximum frequency of visitation of cyclonic storm activity through the entire year, in fact, this area eventually comes under the influence of the majority of cyclonic disturbances which affect the United States. No seasonable variation in precipitation is important, annual rainfall being evenly distributed throughout the months. In general, short period rainfall intensities are greatest in late spring and summer and excessive rains of longer durations occur in late summer and early fall. The extremes of average annual precipitation in New England range from 35 inches in northern Vermont and New Hampshire, with the exception of Mt. Washington where it is considerably higher due to orographic effects in the White Mountains, to 47 inches in southwestern Connecticut and 48 inches on the central coast of Maine. Excessive 24-hour amounts during winter months, December to March, of more than three inches are rare, but summer rainfall excesses are more frequent because of high intensities experienced in thunderstorms. A very even distribution of thunderstorm occurrence makes all parts of the region liable to high short-period rainfall intensities during the summer months.

"Since great accumulations of snow may occur and since the types of cyclonic disturbances which visit this region may result in rapid melting, snow is an extremely important factor in flood production. At Boston, Massachusetts, the average annual snowfall is 43.8 inches, with extremes of 96.4 inches during the winter of 1873-74 and 10 inches during the winter of 1875-76. The average seasonal snowfall at first order Weather Bureau stations ranges from 50.4 inches at

Albany, New York, 60.4 inches at Portland, Maine, and 73.0 inches at Concord, New Hampshire, to 38.6 inches at New Haven, Connecticut, and 32.4 inches at Providence, Rhode Island. The greatest average seasonal snowfall from any reporting station is 168.3 inches at Pittsburgh, New Hampshire."

c. History of Floods.- The only floods on the Contoocook River of which there are any authentic records are the November 1927, the March 1936 and the September 1938. The March 1936 with peak discharge of 13,600 c.f.s. and the September 1938 with peak discharge of 15,400 c.f.s., flood hydrographs at Bennington, New Hampshire, are shown on Plates I-18 and I-19. The November 1927 flood had a peak discharge of only 2,100 c.f.s. on North Branch of Contoocook near Antrim, N. H. and 2,620 c.f.s. on the Blackwater River near Contoocook, N. H. Since these values are not unusually excessive for these drainage areas, the 1927 flood on the Contoocook River could only be considered as the equivalent of a heavy spring freshet.

d. Project Description.- Bennington Reservoir will be formed by a proposed dam on the Contoocook River approximately forty-seven miles above its confluence with the Merrimack River. The dam site is located about one-half mile upstream from the Town of Bennington and approximately eight hundred feet downstream from an existing dam designated as the Powder Mill Dam. The reservoir controls a total drainage area of one hundred eighty-six (186) square miles. Two other proposed flood control reservoirs lie within this area; namely, Mountain Brook Reservoir with a drainage area of fourteen (14) square miles and West Peterboro Reservoir on the Nubanusit Brook with a drainage area of forty-four square miles. In the initial development, the Bennington Reservoir will have a spillway crest at elevation 705, with a storage capacity of 60,000 acre feet. (Area-Capacity Curve-Plate I-3.) The full reservoir covers an area of 3,885 acres and extends a distance of eleven river miles from Bennington to Peterboro. The storage is equivalent to 6.0 inches on the total drainage area of 186 square miles, or 8.8 inches on the net drainage, which area excludes Mountain Brook and West Peterboro Reservoirs. The dam will consist principally of a rolled earth section. The spillway will be a concrete structure of conventional ogee dimensions 300 feet in length, with maximum height above normal tailwater of 40 feet. Six outlets will be located in the spillway section, each with a discharge capacity of approximately 920 c.f.s. with the reservoir level at spillway crest. A common stilling basin will be utilized for both the spillway and outlet discharges.

e. Basin Characteristics.- The drainage basin of the Bennington Reservoir covers 186 square miles in the upper or southern portion of the Contoocook River watershed. It has a maximum width of approximately 12-1/2 miles and a maximum length of approximately 18 miles.

(See Plate I-1.) The headwater tributaries originate on hilly and mountainous slopes that rise 1,200 to 1,500 feet above mean sea level with isolated peaks exceeding 2,000 feet. The reach of river included in the reservoir site is quite flat and consists mainly of undeveloped woodland and meadows. The normal water rise from the dam site to a low timber crib dam located ten miles upstream at North Village is only 25 feet. With spillway crest at elevation 705, the reservoir will cover this North Village Dam and extend approximately another mile to the tailwater of a small dam in the center of Peterboro. Upstream from Peterboro the slope of the river increases considerably and in the 8 miles distance to East Jaffrey, the rise is approximately 290 feet. Above East Jaffrey, the slope flattens again and in this reach is located Contoocook Lake, a low head conservation lake used for downstream regulation. The largest tributary to the Contoocook River is Nubanusit Brook which enters the Contoocook River at Peterboro, and which has a total drainage area of 49 square miles. The proposed West Peterboro Reservoir will control 44 square miles of this drainage area. The tributaries above the Bennington dam site with drainage area exceeding 10 square miles are tabulated below:

Tributary	Enters From	Drainage Area at Mouth (Sq.Mi.)	Mileage at Confluence with Contoocook River (zero mileage at Newburyport Light)
Contoocook Lake	-	15	167.8
Mountain Brook	Left	14	167.7
Gridley River	Right	12	162.3
Meadow Brook	Right	12	161.6
Nubanusit Brook	Left	49	158.7
Boglie Brook	Right	13	155.4
Otter Brook	Right	16	153.6
Ferguson Brook	Left	12	151.9
Moose Brook	Left	14	149.4

It should be noted that the last four brooks: Boglie, Otter, Ferguson and Moose Brooks all enter directly into the reservoir area without any distance of flow in the Contoocook River. This factor is important because, where the flows from the tributaries were formerly affected by the valley storage in the main river, the run-off now goes directly into the reservoir storage with a resulting short period of concentration. The 11-mile reach of the reservoir from Bennington to Peterboro drains a total area of 60 square miles with all tributaries discharging directly into the reservoir. Above Peterboro, or upstream from the upper end of the reservoir, 58 square miles of the remaining 126 square miles of drainage area will be controlled by Mountain Brook and

West Peterboro Reservoirs. This leaves a net of 68 square miles that produces the uncontrolled reservoir inflow from the Contoocook River above Peterboro.

f. Description of Proposed Upstream Reservoirs.-

(1) Mountain Brook Reservoir.- Mountain Brook Reservoir will be located on Mountain Brook (Plate I-1), approximately one mile upstream from East Jaffrey. The reservoir will control a drainage area of 14 square miles and will be principally beneficial for reducing flood flows in East Jaffrey. With spillway crest at elevation 1,027, the reservoir will cover an area of 380 acres and will have a storage capacity of 5,300 acre feet, which is equivalent to 7.1 inches on the drainage area. The dam will consist of a rolled earth section, with an ungated conduit capable of discharging approximately 400 c.f.s. with water surface at spillway crest. The spillway will consist of a small overflow weir at elevation 1,027 discharging into a concrete-lined channel chute extending into a tailwater stilling basin. The spillway design flood has an inflow peak of 24,000 c.f.s. and a spillway design discharge of 14,400 c.f.s.

(2) West Peterboro Reservoir.- West Peterboro Reservoir will be located on Nubanusit Brook, a tributary stream that enters the Contoocook River in the Town of Peterboro. The reservoir will control a drainage area of 44 square miles. The reservoir will cover an area of 830 acres at spillway crest elevation 946 and will have a storage capacity of 12,800 acre feet, which is equivalent to 5.5 inches of storage. The dam at West Peterboro will be a rolled earth section. A gated conduit will provide a maximum discharge capacity of approximately 1,000 c.f.s. which is the maximum downstream channel capacity. The spillway will consist of a small concrete weir at crest elevation 946 located in a rock-cut channel excavated through a saddle in the hills near Half-Moon Pond. The spillway discharge will flow down Ferguson Brook into Bennington Reservoir, a distance of only one and one-half miles.

g. Stream Flow Data.- The stream flow records within the basin are rather meager. Chain-gage measurements were obtained on the Contoocook River at Elmwood (drainage area 168 square miles) from 1917 to 1924, and on the Nubanusit Brook (drainage area 48.1 square miles) from 1920 to 1931. No floods occurred on these tributaries during these periods of record; consequently, no use could be made of the records for unit hydrograph or flood routing studies. For the past two years, Bristol Gages have been operated by the U. S. Geological Survey at North Village, Peterboro, New Hampshire, on the Contoocook River (drainage area 120 square miles) and at Peterboro, New Hampshire, on Nubanusit Brook (drainage area 44 square miles), but since no satisfactory rating curves are available for these gages,

their principal value is in obtaining time of peaking from the recorded stage graphs. Some peak records are available at various dams in the basin for the March 1936, the September 1938 and the June 1944 floods, but these records are complicated by gate operation, partial failure of flashboards, and in some cases, by abutment washouts. The storms of 24 June and 14 September 1944, provided the most accurate and comprehensive set of data for a study of the hydraulics and the hydrology of the Bennington drainage basin. Discharge records of nearby stations in the Contoocook River drainage basin were utilized for volume studies. The following table summarizes the discharge stations used in estimating stream flow data:

Station	River	Type	D.A.	Maximum Discharge of Record	Discharge June 1944
Peterboro, N.H.*	Nubanusit	Recording	48.1	1,130	++
Whites Mill Dam	Nubanusit	Bristol Gage	44.0	4,140	1,500
Antrim, N. H.	N.Br.Contoocook	Recording	54.8	4,680	1,290
Jackman, N. H.	Boards Brook	Bristol Gage	56	++	3,300
Davisville, N.H.	Warner	Recording	146	++	3,940
N.Village Dam	Contoocook	Bristol Gage	120	10,400	++
Elmwood*	Contoocook	Chain	168	4,720	++
Powder Mill Dam+	Contoocook	Non-Recording	186	15,400	5,200
W. Henniker	Contoocook	Recording	368	22,200	8,700

* This station is discontinued. Maximum discharge during period of record.
\+ This dam is located at the proposed Bennington Dam site.
++ No record.

For general information, a long term hydrograph (1917-1942 inclusive) has been constructed and is shown on Plates I-5, I-6 and I-7. The hydrograph was constructed as follows: The record on the Contoocook River at Elmwood was increased in proportion to drainage areas (186 ÷ 168) to provide the first seven years of record at the Bennington dam site. Then, as this station was discontinued in 1924, it was necessary to develop data for 1924 to 1939 by pro-rating available data from the adjacent drainage area of the Souhegan River at Merrimack, N. H. (186 ÷ 171). In October 1939, a new gaging station was installed near Henniker, N. H., and this station was utilized in conjunction with the records of the North Branch near Antrim to furnish the balance of the record. The Antrim flow was deducted from the flow at Henniker, and the difference was pro-rated to give the flow at the Bennington Dam site. In addition to this, a short table

is listed below showing the range of flows from the average annual flow through the Spillway Design Flood.

Comparison of Discharges at Bennington Dam

Average yearly flow	300 c.f.s.
Average annual flood	3,100 c.f.s
Flood of Nov. 5, 1927 (estimated)	5,500 c.f.s.
Flood of Mar. 19, 1936	13,600 c.f.s.
Flood of Sept. 21, 1938	15,400 c.f.s.
Spillway design flood	64,000 c.f.s.
Spillway design flood, inflow to reservoir	77,600 c.f.s

h. Precipitation Records.- The basin is fairly well covered by precipitation stations as illustrated on Plate I-2. The stations at Surry Mountain Dam and at Hillsboro are automatic recorders. The stations at Greenville, Peterboro and Fitzwilliam, New Hampshire are non-recorders and read daily. The automatic recorders are recent installations with short-term records, but since most of the Contoocook River studies have been based on the June 1944 storm, these records have been very helpful. All of these stations are reported in the Hydrologic Bulletin for the North Atlantic District, published by the Weather Bureau in cooperation with the Corps of Engineers.

For general information, a table of rainfall stations showing normal monthly and average annual precipitation is given below, (see Table I). In addition, a table of the normal monthly and average annual temperatures for these stations, where available, is given, (see Table II). To supplement this information, a complete table of comparative data and extremes covering the climatology of the U. S. Weather Bureau of Concord, N. H., is given in Table III.

TABLE I. PRECIPITATION - INCHES

Station	Fitzwilliam	Franklin	Keene	Manchester	Nashua	Newport
Years of Record	21 years Average Monthly	39 years Average Monthly	49 years Average Monthly	66 years Average Monthly	57 years Average Monthly	12 years Average Monthly
January	3.21	3.03	2.89	3.29	3.43	3.26
February	2.64	2.70	2.68	2.96	3.32	2.26
March	3.63	3.33	3.21	3.61	3.70	3.65
April	3.71	3.53	3.09	3.19	3.32	3.77
May	3.30	3.19	3.11	3.10	3.00	3.01
June	4.36	3.73	3.23	3.20	3.13	3.63
July	4.24	3.77	3.80	3.43	3.38	3.64
August	3.97	3.47	3.86	3.39	3.53	3.37
September	4.15	3.99	3.60	3.38	3.42	3.78
October	3.20	2.88	2.76	3.02	3.00	2.57
November	3.88	3.25	3.03	3.33	3.25	3.01
December	3.13	3.05	2.99	3.25	3.34	2.76
Average Annual	43.42	39.92	38.25	39.15	39.82	38.71

TABLE II. TEMPERATURE - FAHRENHEIT

Station	Franklin	Keene	Manchester	Nashua
Years of Record	40 years Average Monthly Normal	48 years Average Monthly Normal	12 years Average Monthly Normal	15 years Average Monthly Normal
January	19.9	21.1	24.1	24.2
February	20.0	21.2	23.9	24.4
March	30.9	32.0	32.9	33.3
April	42.9	43.7	43.6	43.8
May	55.1	55.3	56.4	56.3
June	63.9	63.6	65.8	65.2
July	69.5	68.8	70.3	70.2
August	66.6	66.3	68.8	68.2
September	59.4	59.4	60.5	60.8
October	48.1	48.6	49.0	49.8
November	35.9	36.7	38.9	39.1
December	23.9	24.8	27.4	27.3
Annual Normal	44.7	45.1	46.8	46.9

TABLE III.

CLIMATOLOGICAL DATA FROM OBSERVATIONS AT
U. S. WEATHER BUREAU, CONCORD, N. H.

Month	TEMPERATURE					PRECIPITATION IN INCHES		UNMELTED SNOWFALL IN INCHES	
	Mean Max.	Mean Min.	Mean Monthly	Extremes Highest	Extremes Lowest	Monthly Mean	Greatest in 24 hrs.	Monthly Mean	Greatest in 24 hrs.
Length of Record Years	74	74	74	74	74	91	74	74	42
Jan.	29.2	8.9	19.0	72	-35	2.97	2.10	17.6	19.0
Feb.	31.0	9.6	20.3	68	-37	2.65	2.06	17.3	15.0
Mar.	38.4	19.8	29.1	82	-16	3.15	2.59	11.9	12.9
Apr.	52.5	30.6	41.6	92	7	2.96	2.37	4.6	18.3
May	64.8	40.9	52.9	98	22	3.07	3.05	0.1	2.6
June	73.5	49.9	61.7	101	32	3.23	4.47	T	T
July	79.0	55.9	67.4	102	38	3.70	5.11	0	0
Aug.	76.4	54.6	65.5	99	33	3.50	3.32	0	0
Sept.	69.0	47.1	58.0	96	20	3.48	5.97	T	T
Oct.	58.5	37.1	47.8	92	16	3.26	3.45	0.1	1.0
Nov.	44.4	26.4	35.4	80	-17	3.29	4.04	5.4	13.3
Dec.	32.9	15.2	24.0	65	-24	2.92	2.43	11.9	9.5
Year	54.2	33.0	43.6	102	-37	38.18	5.97	68.9	19.0

THE SPILLWAY DESIGN FLOOD

i. General.- Conforming to previous engineering criteria for the design requirements of spillways, a study has been made of the hydrology and hydraulics of the Contoocook River in order to determine the maximum possible flood. The effect of the proposed Mountain Brook and West Peterboro Reservoirs is entirely neglected in this study, an omission that tends towards additional conservatism and safety, but not to the extent that might be anticipated. This factor is discussed further in paragraph t. (4).

j. Reservoir Routing Assumptions.- It is assumed that the reservoir will be filled to normal maximum pool elevation of 705 feet M.S.L., the crest of the spillway, at the beginning of the spillway flood. The outlets are assumed to be inoperative. The spillway rating curve (Plate III-5) was computed for a free overfall ogee spillway using the weir formula $Q = CLH^{3/2}$, where "L" is 300 feet and values of "C" varied to a maximum of 3.8 for the design head. The unit hydrographs are assumed to apply to reservoir inflow, and, consequently, the spillway floods are routed through the reservoir using the gross surcharge storage. It is further assumed that Mountain Brook and West Peterboro Reservoirs are either not constructed or that they are similar to existing lakes and reservoirs and may be neglected in the hydrologic studies.

k. Unit Hydrograph.- Data available for unit hydrograph construction are meager and of only fair accuracy. In view of the ultimate results to be obtained from these data, however, they are considered adequate for the purpose. An outflow unit graph was computed using data collected during the storm of 24 June 1944 (Plates I-8 and I-9). Good discharge records defining the time of peak discharge as well as readings on the rising and falling side of the hydrograph were obtained from the dams in the Town of Bennington, New Hampshire, controlled by the Monadnock Paper Co. The volume of the estimated hydrograph was checked with the observed hydrograph obtained at the U. S. Geological Survey Station at West Henniker, New Hampshire. Good rainfall records were obtained from the stations which are shown together with a summary of the data on Plate I-9. Another outflow unit graph was computed using the estimated 1938 hydrograph which in turn was constructed from the known time of peak and the peak discharge as computed from the observed flood profile and volume comparisons of adjacent stations in the basin (Plate I-11). The rainfall stations used in this study were North Village and Fitzwillian, New Hampshire. The two computed outflow unit graphs showed good agreement in shape

and magnitude, as is indicated on Plates I-9 and I-11. The computed unit hydrographs for Bennington are definitely outflow unit hydrographs that show the effect of the extensive valley storage in the reservoir reach. An estimate of the volume of valley storage obtained during the September 1938 flood was made from survey data and a discharge valley storage curve was constructed. (See paragraph p.) The 1938 flood hydrograph was then back-routed through the reservoir and an estimated inflow hydrograph obtained. A unit graph for the estimated inflow hydrograph was then derived. It is considered that this unit graph is more representative of the conditions to be expected during a flood of spillway design storm magnitude, although still somewhat slower and less peaked, than that resulting from the higher rainfall values obtained during the storm producing the spillway design flood. In order to peak the inflow unit graph of the September 1938 flood to satisfy the high rainfall increments, resort was made to the empirical formulae of Franklin F. Snyder which were treated in his article on "Synthetic Unit Graphs", published in the Transaction of the American Geophysical Union, Part 1, 1938. These fundamental formulae for the analysis of the unit hydrograph are as follows:

$$t_p = C_t \, (L_{ca} L)^{0.3} \qquad (1)$$

$$q_p = C_p \, 640/tp \qquad (2)$$

The nomenclature corresponds to the established symbols for hydrograph study and are briefly described in the tabulation on Plate I-12. In analyzing flood hydrographs of record, the values of L_{ca} and L are obtained from the topographic maps, values of t_p and q_p are obtained from the records of precipitation, and discharge records. In the application of these formulae to the present situation, the constants C_t and C_p are computed from the known unit graphs and then modified to increase the unit discharge and decrease the time of lag, to produce a unit graph applicable to the high rainfall increments of the spillway design flood. Three pairs of coefficients were used, one pair computed and two pairs assumed, to provide three unit hydrographs of various concentrations and peak discharges as shown on Plate I-12. This procedure was followed to give a range in computed spillway floods and to determine the effect of various unit hydrographs on the spillway requirements. The assumed coefficients are tabulated as follows: for unit graph #2, $C_t = 2.62$, $C_p = .400$, and for unit graph #3, $C_t = 2.14$ and $C_p = .500$.

l. Maximum Storm.- Studies previously made of summer-fall and winter-spring rainfall values have shown that for drainage areas of comparable size in this basin, a more severe spillway flood is obtained from summer-fall limiting rates of rainfall, and, consequently, this type of storm has been used in computing spillway floods for Bennington Reservoir. The rainfall intensity curve used for this study (Plate I-13) was based on the maximum possible rainfall for the Contoocook River Basin as determined by the Hydro-Meteorological Section of the U. S. Weather Bureau. The flood producing storm is of 24-hour duration with a total rainfall of 17.4 inches. The distribution of rainfall and run-off for these two storms is shown graphically on Plates I-15 and I-16. The values of precipitation used for this area in terms of 3-hour amounts and in order of magnitude are tabulated below:

3-hour period	Precipitation in Inches
1	7.5
2	6.4
3	2.2
4	0.6
5	0.3
6	0.2
7	0.1
8	0.1
Total Precipitation	17.4

m. Basic Spillway Flood.- The basic spillway flood was computed by applying the basic unit hydrograph shown on Plate I-12 to the rainfall values summarized in the preceding paragraph. An infiltration rate of 0.05 inch per hour and a base flow of 5 c.f.s. per square mile were used. The assumed minimum infiltration rate of 0.05 inch per hour or 0.15 inch per 3-hour period was based on the smallest rate determined from analyzing floods of record for unit hydrographs in this area. The hydrograph of the basic spillway flood (Curve A) and the pluviograph are shown on Plate I-15. The periods of rainfall are arranged in an order that results in the highest peak discharge when computed with the basic unit hydrograph.

n. Variation in Shape and Peak of Basic Spillway Flood.- As the basic spillway flood is derived from an empirical basic unit hydrograph, and a theoretical flood producing storm, further studies were made to determine the effect on the spillway requirements of varying the shape of the spillway flood. Two adjusted unit graphs were constructed as described in paragraph c., and these values were applied to the two

excessive 3-hour rainfall increments of the basic flood (Flood A) to give Floods B and C, shown on Plate I-15. These floods were then routed to show the effect of increased peaking of a constant volume flood as is illustrated graphically on the same plate. A study was then made increasing the volume of the basic flood to 125%, 150% and 200%. The results plotted on Plate I-15 indicate that the spillway surcharge is quite sensitive to volume variations and only to a minor extent is it sensitive to peak variations.

o. Selected Spillway Design Flood.- After consideration of all the factors that enter into the development of an adequate spillway flood, it was decided to adopt the Basic Spillway Flood, as modified by the unit hydrograph #2 (Plate I-12) as the selected spillway flood. This flood is shown on Plate I-15 as hydrograph "B," and summarized in detail on Plate I-16. The pertinent data relative to this design flood are summarized as follows:

Rainfall in inches in 24 hours	17.40
Rate of Infiltration (inches per hour)	0.05
Run-off in inches	16.30
Run-off volume in acre feet	162,800
Peak Inflow in c.f.s.	77,600
Peak Spillway Discharge in c.f.s.	45,900
Maximum Water Surface (ft. above M.S.L.)	716.8
Surcharge Storage Utilized, acre feet	52,000
Surcharge Storage Utilized, inches	5.24

The spillway design flood provides a safety factor of approximately 35% over the basic spillway flood as shown on Plate I-15.

p. Effect of Valley Storage.- Valley storage in the Bennington reach is very extensive as observed in the recent floods in 1936 and 1938. From flood profiles and topographical surveys of the reservoir area, the valley storage in this reach for the 1938 flood (maximum of record, 15,400 c.f.s. at Bennington) amounted to 10,400 acre feet. A discharge-valley storage curve was constructed based on this computed value and an extrapolation using a computed natural discharge rating curve at the dam site and the reservoir area-capacity curve. Assuming that reservoir routing methods were applicable, the spillway design flood was then routed through this valley storage to obtain the natural spillway design flood hydrograph at the Bennington Dam site. This routing indicated that the peak inflow of 77,600 c.f.s. would be reduced by the natural storage to a peak outflow of approximately 64,000 c.f.s. The Myers Coefficient "C" in the flood peak relationship, $Q = C \cdot \sqrt{\text{Drainage Area}}$, and peak discharge in c.f.s. per square mile are as follows:

	"C"	Maximum c.f.s. per sq. mi.
Inflow 77,600 c.f.s.	5,700	417
Outflow 64,000 c.f.s.	4,700	344

q. Freeboard.- The theoretical freeboard required for Bennington Reservoir, based on the criteria outlined in the Engineer Bulletin R. & H. No. 9, 1938, is 7.5 feet. This height was computed from the following data:

Fetch in miles	= 5.5
Wind velocity in miles per hour	= 80
Angle of wind and fetch	= 0
Depth of water in feet	= 40

r. Top of Dam.- The top elevation of the Bennington Dam was determined as follows:

Elevation, crest of spillway	705.0
Maximum head on spillway from spillway design flood	11.8
Freeboard requirement	7.5
	724.3
Adopted elevation for top of dam	724.0

The total reservoir storage capacity is 151,500 acre feet distributed between various stages as follows:

	Acre Feet	Inches
To Spillway crest, elev. 705	60,000	6.0
Maximum surcharge, elev. 705-716.8	52,000	5.2
Freeboard, elev. 716.8-724.0	39,500	4.0
	151,500	15.2

s. Top of Dam - Ultimate Development.- If, in the future, in the interests of conservation storage, it is desired to raise the spillway from elevation 705.0 to 712.0, the top elevation of dam is determined as follows:

Elevation, crest of spillway	712.0
Maximum head on spillway from spillway design flood	11.1
Freeboard requirement	7.3
	730.4
Adopted elevation for top of dam	730.0

The justification for reducing the theoretical height of dam to the nearest foot is similar to that described for the initial scheme in the next paragraph. The head on the ultimate

spillway of 11.1 feet was arrived at by routing the selected spillway flood peak inflow 77,600 c.f.s. through the increased surcharge storage obtaining a peak spillway discharge of 42,200 c.f.s.

t. Discussion Concerning Selected Spillway Design Flood.-

(1) The most uncertain factor in the construction of the spillway floods is the unit hydrograph that is applicable to represent the summation of all the various inflows that enter into the reservoir. The discharge in the Contoocook River entering the reservoir at Peterboro represents the run-off from 120 square miles, or less than 2/3 of the total drainage area. The remainder, or 66 square miles, consists of many small tributaries entering the reservoir from both sides and the reservoir area itself. It is impossible to evaluate accurately all these various points of inflows to obtain a true unit hydrograph, but it is believed that the estimated basic unit graph is conservative, inasmuch as it has been derived from the highest flood of record at the dam site and adjusted by back-routing through the natural valley storage to obtain an approximation of the true inflow hydrograph to the reservoir. The modification of this basic unit graph to obtain a unit graph applicable for use with the higher rainfall values, i.e. over 2.5 inches in 3 hours, was made from an adjustment of the Snyder peaking and lagging coefficients for the basic unit graph. Since careful consideration has been given to the influence of the unit graph in peaking the spillway design flood and since the surcharge storage of the reservoir is so great that the pool elevation is more sensitive to volume fluctuations (see Plate I-15) than to peak considerations, it is felt that the unit graphs used are sufficiently conservative.

(2) The possibility of the adopted maximum rainfall values being greatly exceeded on this drainage area is considered too remote to require a factor of safety for more severe rainfall. The effect of increasing the volume of the flood hydrograph has more effect on the height of surcharge than peaking the hydrograph with constant volume, but it is concluded that any assurance factor for greater storm run-off is amply provided by the freeboard requirements.

(3) The theoretical top of dam should be elevation 724.3; however, it is believed that the establishment of elevation 724.0 is permissible due to (1) the extreme improbability of all design criteria occurring simultaneously, that is,

full reservoir at the beginning of the flood, outlets inoperative and hurricane wind at peak reservoir stage, and (2) the reduced effect of Mountain Brook and West Peterboro Reservoirs on the design flood as described in the following paragraph.

(4) Although Mountain Brook and West Peterboro Reservoirs have been neglected in determining the basic spillway requirements for Bennington Reservoir, a study has been made to ascertain the effect of these two upstream reservoirs during the spillway design flood. Such an analysis of these reservoirs is difficult, for the adopted unit graphs utilized in deriving the spillway design floods for each particular reservoir are based on synthetic methods using selected coefficients, hence the unit graphs are not correlated necessarily with each other. Consequently, instead of basing the modified inflow to the Bennington Reservoir on the summation of the Mountain Brook and West Peterboro spillway discharges added to the run-off from the 128 square miles of uncontrolled area, the analysis has been made on the basis of determining the differences in the probable discharges from Mountain Brook and Nubanusit Brook with and without the respective reservoirs and then modifying the selected spillway design flood for the Bennington Reservoir by these differences. This method is shown on Plate I-17. Inflow hydrographs were constructed for both the Mountain Brook and West Peterboro Reservoirs using the limiting rainfall values for 186 square miles and their respective adopted unit graphs. These natural inflows were then considered as occurring (1) without any flood control dams and (2) with the dams. The natural and modified inflows to the Bennington Dam were determined from these natural and spillway discharges from the reservoirs. The shaded areas on these hydrographs represent the differences in the natural and modified inflow to Bennington Reservoir and consequently the ordinates of the selected Bennington spillway design flood were modified by the ordinates of the shaded areas. The effect of Mountain Brook and West Peterboro is to reduce the peak inflow to the Bennington Reservoir from 77,600 c.f.s. to 68,000 c.f.s., the spillway discharge from 45,900 c.f.s. to 42,600 c.f.s., and the reservoir stage from elevation 716.8 to 716.2.

u. Downstream Channel Capacity.- It has been determined from computations on downstream dams and from field observations of actual flood conditions that the safe channel capacity for the reaches below the proposed dam is approximately 4,000 c.f.s. The existing dams can safely discharge higher flows, but the river below Bennington is flat and sluggish and floods greater than 4,000 c.f.s. inundate low areas and endanger some of the adjacent highways. A storm in June 1944 caused an estimated discharge at Bennington of 5,100 c.f.s. No material damage other than lost flashboards occurred at Bennington, but downstream the river had overflowed its banks and made some of the roads impassable.

v. Proposed Method of Operation.- Six gated outlets are provided for controlling normal river flow and floods. These outlets are located in the concrete spillway section and discharge into the stilling basin of the spillway. Each conduit is 6'-0" high and 4'-0" wide and has an invert elevation of 667.0, which is 1.5 feet above the flashboard elevation of the Monadnock Power Dam about 1/4 mile downstream. The present conservation pond level maintained upstream by the existing Power Mill Dam will be continued by gate operation. As the top of flashboards at the Powder Mill Dam is elevation 678.15, the conservation drawdown depth will range from conduit invert Elev. 667.0 to 678.2. Normally all gates will be closed except as required for regulating daily flow for the Bennington power dams. Should a storm occur on the Bennington Reservoir drainage area to produce freshet or flood flows and cause the reservoir level to rise above elevation 678.2, the gates will be opened in the following sequence:

Reservoir stage, 678.2 - 1 gate open, storage 0
Reservoir stage, 680 - 2 gates open, storage 1,000 a.f.
Reservoir stage, 682 - 3 gates open, storage 3,000 a.f.
Reservoir stage, 684 - 4 gates open, storage 5,000 a.f.

The discharge capacity of each conduit in this range (see Plate III-1) is 400 c.f.s. with pool elevation 678, and 570 c.f.s. with pool at elevation 684. Hence, the total capacity of 4 gates open with reservoir stage 684 is 2,280 c.f.s. At full pool, spillway crest elevation 705, the discharge capacity of the 4 conduits is approximately 3,700 c.f.s. The two additional gates will be used for emergency purposes and for emptying the reservoir following a flood. The additional storage provided by Mountain Brook and West Peterboro safely permits

flexibility in gate operation to further decrease the downstream flows, and consequently augment the flood control benefits. If the flood is of such magnitude that the discharge from the uncontrolled tributaries downstream from the Bennington Dam exceeds the safe discharge capacity of the Contoocook River, it is proposed to close two more gates, thus leaving two open, in the Bennington Dam to decrease the discharge. Ordinarily this procedure will reduce the discharge from about 3,000 to 1,500 c.f.s. This proposed method of operation is interrelated with a contemplated scheme for all the reservoirs in the Merrimack River Basin in which it is believed that more complete flood control can be obtained by reservoir operation to desynchronize flood flows from the reservoirs and the uncontrolled tributaries. This method necessarily will require a carefully coordinated system of communications and weather forecasting. Operational experience may indicate in the future that some modification of this proposed method may be desirable, but it is believed that the six gated outlets will provide sufficient flexibility for any operating criteria.

w. Flood Control Storage and Reservoir Effectiveness.- If the Bennington Reservoir is utilized as a simple retarding basin with four gates open throughout the flood period in conjunction with Mountain Brook and West Peterboro Reservoirs, it has been determined from routing the reservoir design flood (March 1936 flood) that only 50,000 acre-feet of storage are utilized. The outlet discharge during the peak of the flood is approximately 3,000 c.f.s. The proposed method of operation, however, as described in the preceding paragraph, is based on reducing the reservoir discharge during the peak of the flood to obtain additional downstream flood control benefits (see Plates I-18 and I-19). The storage necessary to safely control the reservoir design flood with this proposed method of gate operation is determined to be approximately 60,000 acre-feet, which results in a spillway crest at elevation 705. The effective controlled discharge during the flood peak is approximately 1,500 c.f.s. The flood control benefits to be gained by this gate operation are realized the entire length of the Contoocook and Merrimack Rivers because the conduits discharge at approximately a constant rate of flow that is effective in all the lower reaches. For example, at Manchester on the Merrimack River, the Bennington Reservoir when used as a simple retarding basin with four gates open, would reduce the peak of the 1936 flood approximately 6,000 c.f.s. With the proposed gate operation during the peak of the flood, this may be further reduced by approximately 1,200 c.f.s. or a total reduction of 7,200 c.f.s., hence making the Bennington Reservoir about 20 per cent more effective at Manchester with the flexible gate operation scheme.

In order to provide 60,000 acre-feet of storage in place of 50,000 acre-feet, the required spillway crest is at elevation 705.0 instead of 702.5, or a difference in height of 2.5 feet. The following tabulation indicates the cost analysis of constructing dams to these two elevations and the relative cost of the flood control storage:

Spillway Crest	Total Cost	Storage	Cost per Acre-Foot
Elev. 705.0	$3,880,000	60,000 A.F.	$64.77
702.5	$3,750,000	50,000 A.F.	$75.00
Difference	$ 130,000	10,000 A.F.	$13.00

The small difference in total costs is due to the fact that the construction of a dam to either elevation involves practically the same amount of land acquisition, highway relocations, and modifications in utilities. Consequently, the differential represents structural costs of the dam and its appurtenances almost entirely. The small additional cost for 10,000 acre-feet of storage to provide flexibility of reservoir operation with the resulting increase in downstream control justifies the selection of crest elevation 705 for the initial development. In the ultimate development, it is proposed to provide only 50,000 acre-feet for flood control instead of the 60,000 acre-feet provided in the initial development. The reduction in specified flood control storage is justified on the basis of the following:

(1) The conservation pool provides a constant head on the conduits and hence allows a higher rate of reservoir discharge during the early part of a flood.

(2) The probability that the conservation pool is not full at the beginning of a flood thus permitting some of the conservation storage to be utilized for flood control.

Comparative results of the allocation of flood control storage in the initial and ultimate developments on the reservoir design flood is discussed further in paragraph x and is illustrated on Plates I-18 and I-20.

x. Effect on 1936 and 1938 Floods.- The two severe floods of record, occurring in March 1936 and September 1938, have been used as reservoir design floods to check the adequacy of the storage, design discharge, and the proposed method of operation. These two floods prove to be ideal examples, for one (1936) is practically a three-peak flood with a large volume of run-off, while the second (1938) is a single-

peak flood with a higher peak discharge than the first. The hydrographs of both floods are based on observed peak discharges with the shape and volume of hydrographs determined from flow records on other comparable rivers. Although both floods were reconstructed, provision was made for the effect of valley storage by back-routing the Bennington hydrograph through the valley storage-discharge relationship to obtain the theoretical inflow to the reservoir reach. The effect of Mountain Brook Reservoir on these floods was disregarded as the drainage area controlled is so small that the effect at Bennington can be neglected. However, the reservoir inflows have been reduced by the effect of West Peterboro Reservoir. The net inflow floods were then routed through the gross storage to obtain the reservoir discharge and stage graphs.

Plates I-18 and I-19 show the effect of the initial development at Bennington on the 1936 and 1938 floods, and Plate I-20 shows the effect of the ultimate development on the 1936 flood. All reservoir discharges are based on the prescribed method of regulation with gate operation during the peak of the flood for desynchronizing the flow from the reservoir with respect to that from the downstream tributaries. This illustrated method provides the optimum utilization of the flood control storage with maximum downstream benefits. No attempt has been made to adjust the gate operation to result in a full reservoir without any spillway discharge, but operation has been based on the assumption that the regulation was determined from the daily development of the flood without definite knowledge of its magnitude or duration. It is to be noted that a small spillway discharge occurred in the routings of the 1936 flood in both the initial and ultimate developments which had no appreciable effect on the downstream flows. The following table summarizes the effect of the reservoir on these floods:

	Initial Development		Ultimate Development
	1936	1938	1936
Natural peak at Bennington in c.f.s.	13,600	15,400	13,600
Reservoir inflow allowing for valley storage in c.f.s.	16,500	18,000	16,500
Reservoir inflow reduced by West Peterboro Reservoir in c.f.s.	14,500	14,700	14,500

	Initial Development		Ultimate Development
	1936	1938	1936
Volume of Bennington hydrograph in inches	12.0	7.4	12.0
Duration of flood in days	13	6	13
Effective Controlled Reservoir discharge during flood	1800	1760	2000
Maximum reservoir discharge in c.f.s. during emptying	4000	4000	4000
Maximum water surface elevation	706.0	702.1	712.9
Flood control storage utilized in acre feet	64,000	49,000	54,000

The comparison and effectiveness of the proposed 50,000 acre feet of flood control storage in the ultimate development with the 60,000 acre feet in the initial is shown also on Plates I-18 and I-20. Due to the higher available head on the conduits in the ultimate scheme the reservoir outflow is greater in the early part of the flood and consequently does not require utilization of storage to build up discharge head. It is to be noted that, although there is some difference in this method of gate operation, the maximum reservoir stage of the initial development is 1.0 feet above the crest of the spillway and only 0.9 feet in the ultimate.

y. Time for Emptying Reservoir.- Plate I-21 shows the time required and the proposed procedure for emptying the reservoir following a flood that has filled the reservoir to spillway crest. It is assumed that the same flood that has filled Bennington Reservoir also has filled Mountain Brook and West Peterboro Reservoirs; hence, the inflow to Bennington during the emptying period includes the discharge from these two reservoirs in addition to the flow from the uncontrolled area. It is assumed that the proposed operation of four gates open will be maintained for several days following the flood crest in order to minimize the flood flows downstream, and to keep the discharge within safe channel capacity. The fifth gate will then be opened followed later by the sixth gate, in order to increase the discharge to the channel capacity. The recovery of available storage following the period of a

full reservoir is shown also on Plate I-21. Two inches of storage will be recovered in approximately 6 days, while half the reservoir capacity (3 inches, 30,000 acre feet) will be available in approximately 8-1/2 days. The reservoir will be empty in 15 days. It is considered that this rate of storage recovery is adequate protection for the possibility of a second flood. This protection is substantiated by the effect on the 1936 flood (Plate I-18) which is practically a three-storm flood. It is obvious that with the inflow assumptions indicated on Plate I-21, the Bennington Reservoir is discharging the total volume stored in Mountain Brook and West Peterboro Reservoirs, as well as that in the Bennington Reservoir, which results in a total of 78,000 acre feet passing through the conduits in the indicated time. In addition to the storage recovered in Bennington Reservoir during the emptying period, similar storage is becoming available in Mountain Brook and West Peterboro Reservoirs.

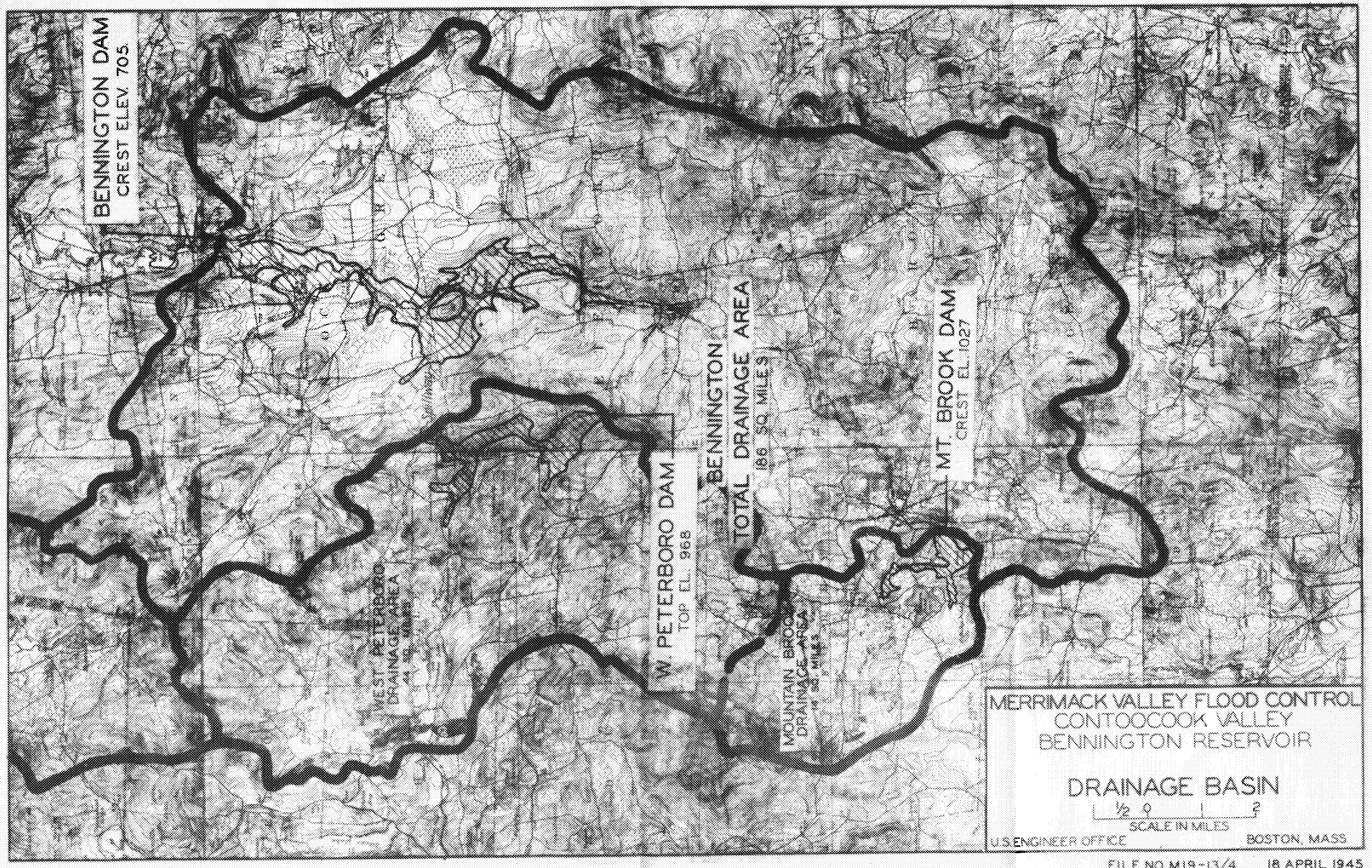
z. Effect of Backwater.- Due to the large expanse of the Bennington Reservoir and the absence of any narrow restricting sections, the backwater effect of the flow through the reservoir is negligible and is not a design consideration of the dam and reservoir. Moreover, it is to be noted on the curve of reservoir stage on Plates I-18 and I-20 that the maximum pool elevation will occur several days following the peak inflow, and consequently the velocity of flow through the reservoir is insignificant. There will be undoubtedly some backwater effect in the water surface during the spillway design flood, but the backwater will be negligible, compared with the area that will be flooded by the unmodified discharges.

aa. Cofferdam Design Flood.- During the construction of the dam, it will be necessary to divert the Contoocook River from its natural channel through the previously built conduits in the overflow section of the dam in order to permit closure of the non-overflow embankment section in the dry. The diversion will be accomplished by the construction of a cofferdam downstream from Powder Mill Dam as indicated on Plate IV-6. In order to determine the height to construct this cofferdam, an inspection was made of the composite hydrograph plotted on Plates I-5, I-6, and I-7. This record of more than a quarter of a century shows that a storm causing a discharge of more than 5,000 c.f.s. at the Bennington dam site occurred only four times. It was decided to route the storm of 6 April 1923, which was the maximum summer flood during the periods of record at Elmwood, (with a peak inflow at dam site 3,200 c.f.s.), and the storm of 24 June 1944 (peak inflow at dam site 5,100 c.f.s.) through the six open conduits

of the dam. The storm of 6 April 1923 when routed gave a pool elevation of 681.0 and the storm of 24 June 1944 when routed reached pool elevation 683.8. It was decided to build the cofferdam to elevation 685 allowing 1.2 feet of fre board over the greater pool elevation. The discharge capacity of the six conduits with pool at elevation 683 is approximately 3270 c.f.s. This height is considered ample to protect the cofferdam against overtopping by any heavy freshet of normal expectancy. The downstream cofferdam with top at elevation 670 provided approximately 2.9 feet of freeboard with a discharge of 3300 c.f.s. assuming the flashboards on the Monadnock Power Dam f il or are released, or about 1.0 foot of freeboard if the boards are not removed.

bb. Stage Frequency Curve (Plate I-22).- A study was made to determine the probable reservoir stage equalled or exceeded in a designated number of years. In this connection, two distinct steps were required: (1) to construct a natural discharge volume frequency curve for the Bennington site, and (2) to construct a stage frequency for the reservoir which is a direct result of the inflow volume. Available stream flow data on the Contoocook River in the vicinity of Bennington, required for the first step, is limited to 7 years of record at Elmwood, N. H. (drainage area - 168 sq. mi.) from October 1917 to September 1924. No large floods occurred during this period. Because of the lack of essential stream flow data on the Contoocook River, it was necessary to utilize records of an adjacent watershed. The Souhegan River at Merrimack, N. H. (drainage area = 171 sq. mi.) with records available from July 1907 up to the present time was selected for this purpose. Comparison of the Elmwood records with the concurrent Souhegan River records indicates that differences of considerable magnitude frequently occur in comparable one day volumes, however, comparable maximum two day volumes from the two drainage areas check very closely. It was also determined that the reservoir stage is dependent principally on volume, and that the type of hydrograph, that is a flash flood or a long flat flood, has little influence on the reservoir stage. It was therefore concluded that stream flow records at Merrimack, N. H., modified for the Bennington drainage area could be utilized. From these data, a two day volume frequency curve was constructed using the formula, $y = M/n - 0.5$, in which Y = the probability of occurrence in years, M = the years of available records, and N = the summation of occurrences. Several floods of computed frequency were then routed through the reservoir

to obtain the maximum reservoir stage. As the final method of gate operation is uncertain, the reservoir routings were based on the assumption that the reservoir was a simple retarding basin with 4 gates open continuously. The results of these computations are plotted graphically on Plate I - 22.



FILE NO. M19-13/4 18 APRIL 1945

PLATE I-1

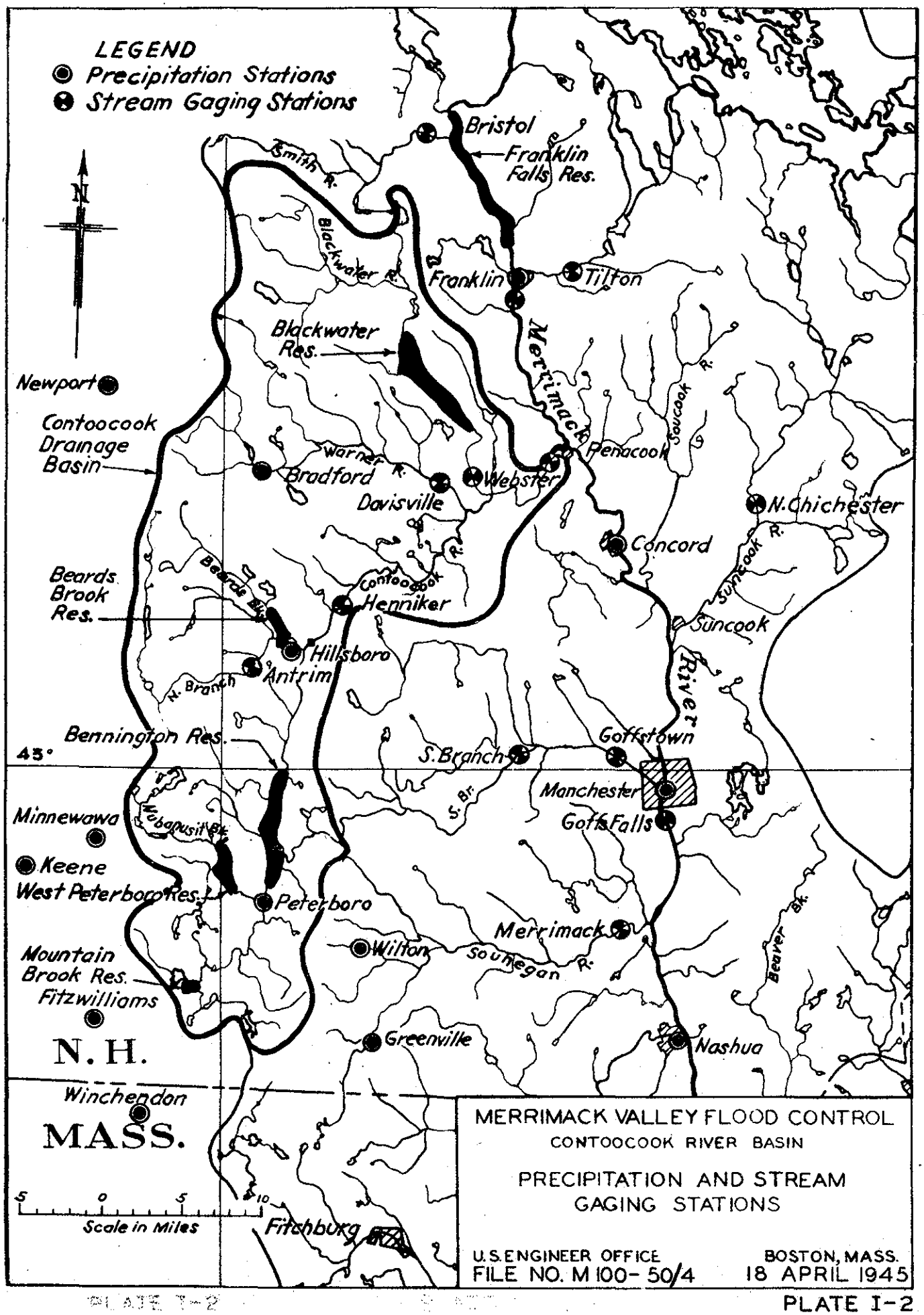
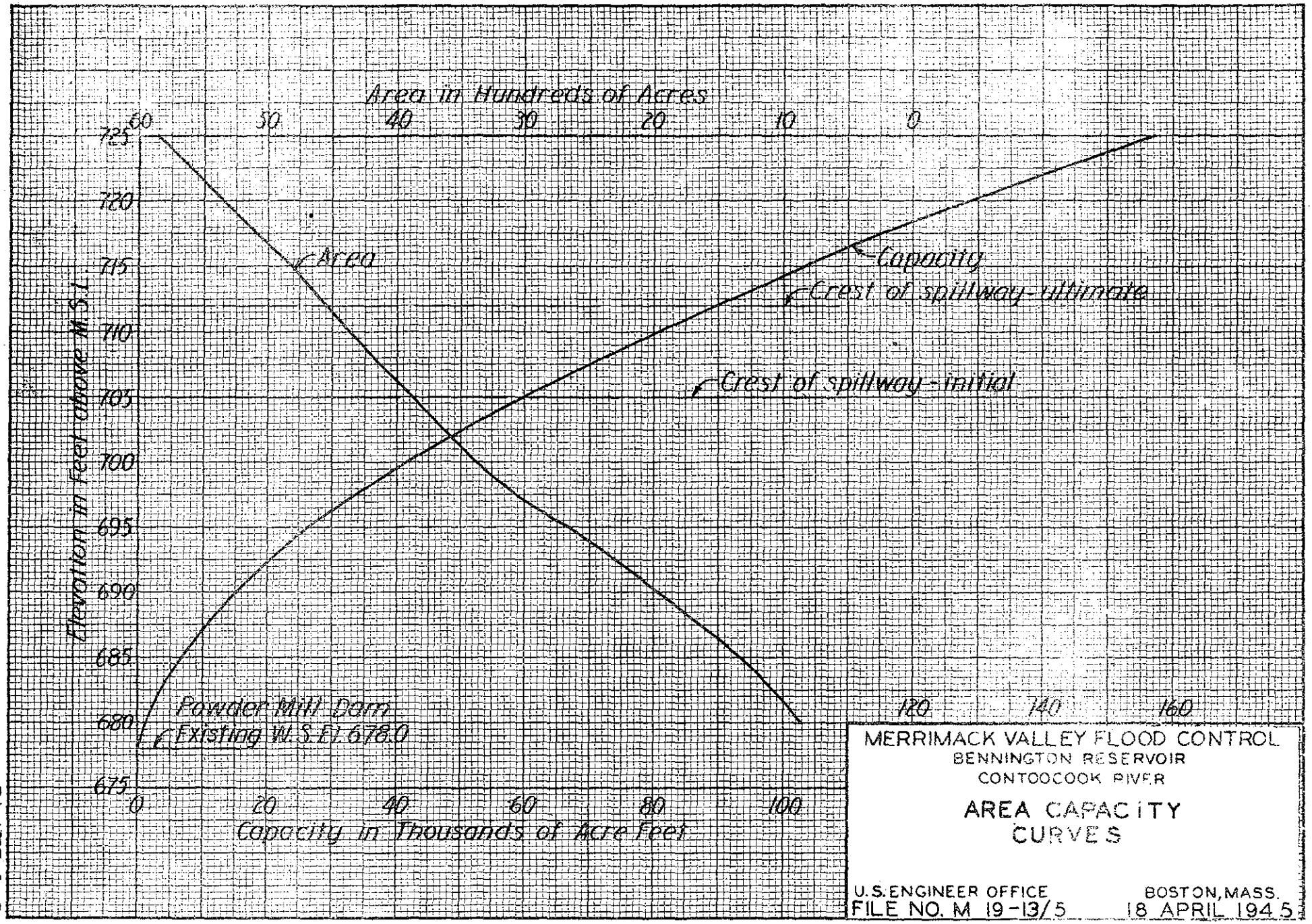

LEGEND
Precipitation Stations
Stream Gaging Stations
N
Bristol
Franklin Falls Res.
Smith R.
Blackwater R.
Franklin
Tilton
Blackwater Res.
Merrimack R.
Newport
Contoocook Drainage Basin
Warner R.
Bradford
Webster
Davisville
Penacook
Soucook R.
N. Chichester
Concord
Suncook R.
Beards Brook Res.
Beards Bk.
Contoocook R.
Henniker
Suncook
Hillsboro
Antrim
N. Branch
Bennington Res.
45°
Goffstown
S. Branch
S. Br.
Manchester
Goffs Falls
Merrimack R.
Minnewawa
Nubanusit Bk.
Keene
West Peterboro Res.
Peterboro
Merrimack
Wilton
Southegan R.
Beaver Bk.
Mountain Brook Res.
Fitzwilliams
N. H.
Greenville
Nashua
Winchendon
MASS.
5 0 5 10
Scale in Miles
Fitchburg
MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK RIVER BASIN
PRECIPITATION AND STREAM GAGING STATIONS
U.S. ENGINEER OFFICE
FILE NO. M 100-50/4
BOSTON, MASS.
18 APRIL 1945

PLATE I-2



MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

AREA CAPACITY CURVES

U.S. ENGINEER OFFICE
FILE NO. M 19-13/5

BOSTON, MASS.
18 APRIL 1945

PLATE I-3

D 1 ATE I - 3

Total drainage area 186 Sq. mi.

Net drainage area 128 Sq. mi.

DRAINAGE AREA VS. DISTANCE

INCREMENT OF DRAINAGE AREA SQ. MILES

0 4 8 12

0 4 8 12 16 20 24 28 32

RIVER MILES ABOVE BENNINGTON, N.H.

Nubanusit Lake

Harrisville Pond

Skatutakee Lake

Nubanusit Brook

Gridley Brook

Mead Brook

Stacy Brook

Contoocook Lake

Mountain Brook

Proposed West Peterboro Reservoir

Proposed Mountain Brook Reservoir

Meadow Brook

Proposed Bennington Reservoir

Powder Mill Dam

ELEVATION IN FEET ABOVE M.S.L.

1400 1200 1000 900 800 700 650

PROFILE

0 2 4 6 8 10 12 14 16 18 20 22 24

RIVER MILES ABOVE HIGHWAY BRIDGE BENNINGTON, N.H.

MERRIMACK VALLEY FLOOD CON.

BENNINGTON RESERVOIR

DRAINAGE BASIN CHARACTERISTICS

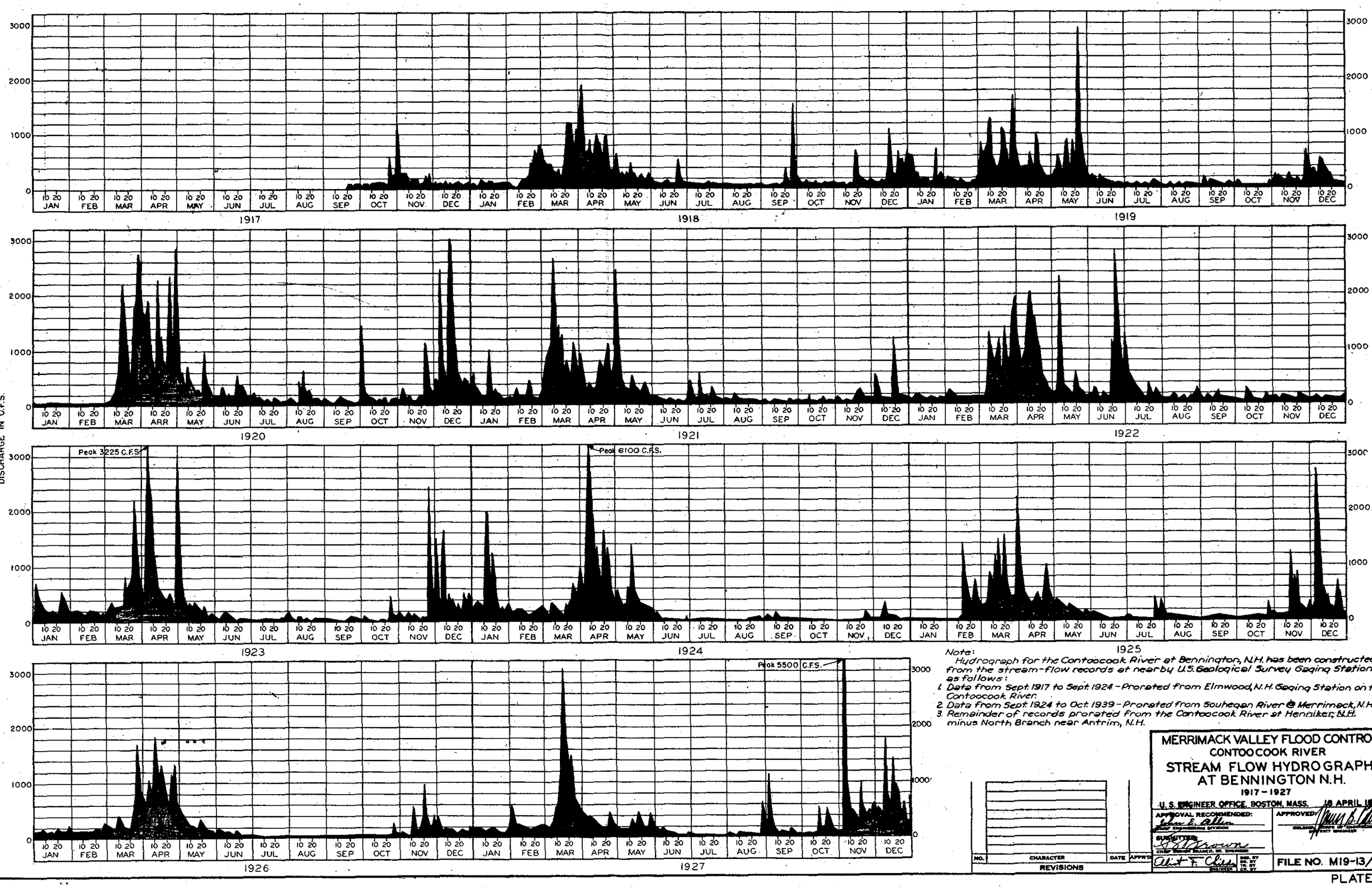
FILE NO. M 19-13/6 18 APR. 194

PLATE I-4

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

DISCHARGE IN C.F.S.



Note:

Hydrograph for the Contoocook River at Bennington, N.H. has been constructed from the stream-flow records at nearby U.S. Geological Survey Gaging Stations as follows:

1. *Data from Sept. 1917 to Sept. 1924 - Prorated from Elmwood, N.H. Gaging Station on the Contoocook River.*
2. *Data from Sept. 1924 to Oct. 1939 - Prorated from Souhegan River @ Merrimack, N.H.*
3. *Remainder of records prorated from the Contoocook River at Henniker, N.H. minus North Branch near Antrim, N.H.*

MERRIMACK VALLEY FLOOD CONTROL

CONTOOCOOK RIVER

STREAM FLOW HYDROGRAPH AT BENNINGTON N.H.

1917 – 1927

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: Chief Engineering Division

APPROVED: District Engineer

SUBMITTED: Chief Drainage & Irr. Br. Engineer

Asst. Engineer

NO.	CHARACTER	DATE	APP'D
	REVISIONS		

DRN. BY
TR. BY
CH. BY

FILE NO. M19-13/7

PLATE I-5

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

DISCHARGE IN C.F.S.

3000 2000 1000 0

10 20 JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC

1928

1929

1930

Peak 3325 C.F.S.

Peak 3500 C.F.S.

1931

1932

1933

Peak 6952 C.F.S.

Mar. 13, Peak 4400 C.F.S.
Mar. 19, Peak 13,500 C.F.S.

Mar. 20, Peak 5281 C.F.S.
Mar. 22, Peak 5841 C.F.S.

1934

1935

1936

Peak 4525 C.F.S.

Peak 4100 C.F.S.

Sept. 22, Peak 15,400 C.F.S.

1937

1938

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK RIVER
STREAM FLOW HYDROGRAPH
AT BENNINGTON N.H.
1928 - 1938

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: Chief Engineering Division

APPROVED: Colonel, Corps of Engineers, District Engineer

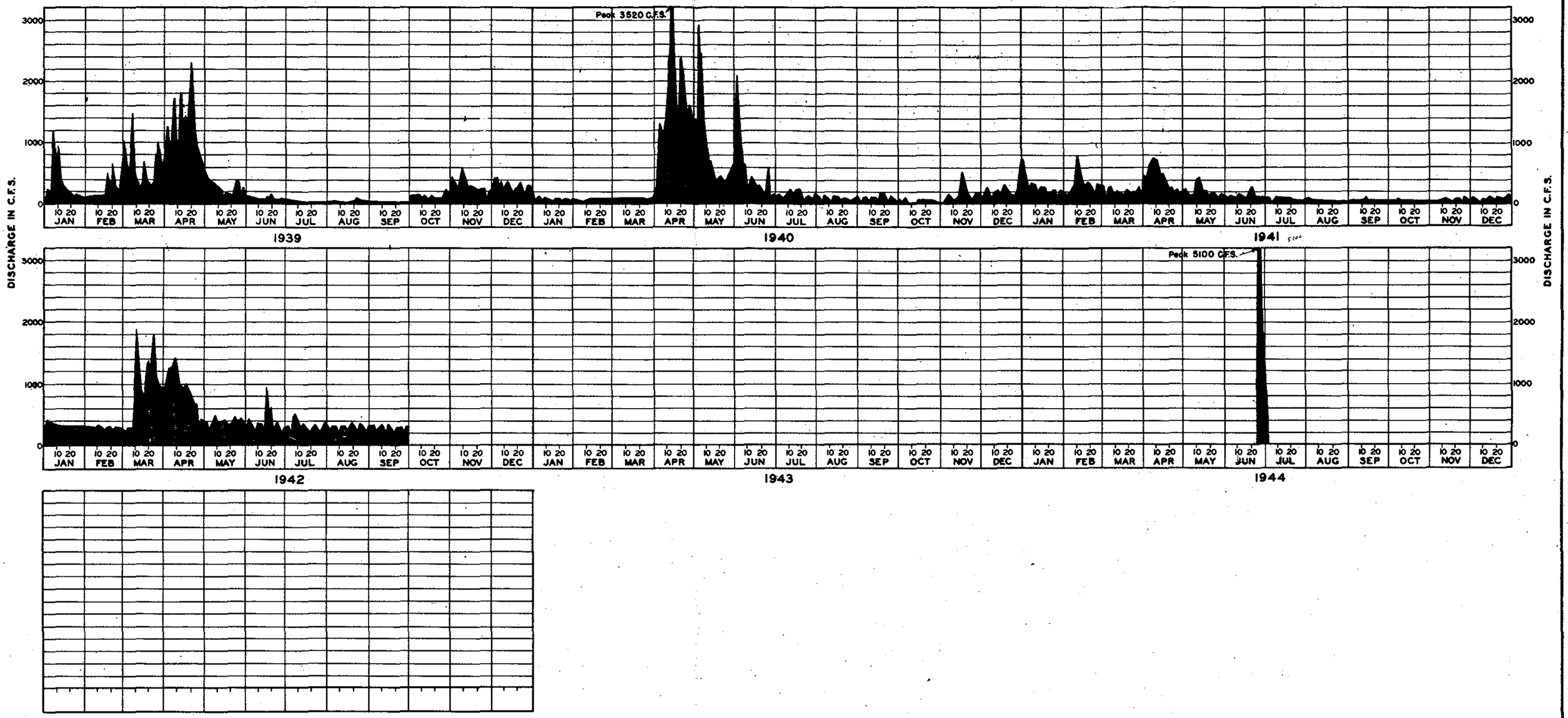
SUBMITTED: Chief Design Branch, Sr. Engineer

Engineer

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M19-13/8

PLATE I-6



MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK RIVER
STREAM FLOW HYDROGRAPH AT BENNINGTON N.H.
1939 - 1942

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945.

APPROVAL RECOMMENDED: John E. Allen, Engineering Division

SUBMITTED: L. B. Brown, District Engineer Council, Bl. Engineer

APPROVED: James B. Stark, Colonel, Corps of Engineers, District Engineer

Alice F. Chase, Engineer

NO.	CHARACTER	DATE	APP'D
	REVISIONS		

DRN. BY
DES. BY
TR. BY
CH. BY

FILE NO. M19-13/9

PLATE I-7

Computed by E.L.P.
Date Feb., 1945

UNIT HYDROGRAPH DETERMINATION

STREAM Contoocook River LOCATION Bennington, N. H.

DRAINAGE AREA 186 SQ. MI.

STORM OF June 24, 1944 PREPARED BY Boston DIST. N.E. DIV.

AV. RAINFALL 5.80 IN.; RAINFALL-EXCESS 2.38 IN.; F_{av} 0.08 IN./HR.

L 20 mi; L_{co} 7.70 mi; $(LL_{co})^{0.3}$ 4.53 ; t_R 3 hrs;

LAG(t_{pR}) 26.5 hrs; C_{tR} 5.78 q_{pR} 10.3 c.f.s./sq.mi.; C_p 640 268

LAG(t_{cmR}) 48.9 hrs; W_{50} 51.9 hrs; W_{75} 29.2 hrs; SLOPE

TIME	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED 6 HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED 3 HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
M	700					
	740	740	0	0	0	790
24 N	900	740	160	330	460	960
	1480	740	740	915	1060	1630
M	3320	750	2520	1465	1590	3520
	4860	750	4110	1835	1880	4730
25 N	5090	750	4340	1910	1910	5180
	4990	750	4240	1820	1760	5100
M	4680	750	3930	1610	1560	4780
	4310	750	3560	1420	1380	4350
26 N	3910	740	3170	1250	1210	3900
	3480	730	2730	1095	1050	3480
M	3100	710	2390	935	900	3090
	2790	700	2090	795	760	2780
27 N	2480	690	1790	670	640	2490
	2230	660	1570	560	530	2230
M	2010	650	1360	455	430	2000
	1830	630	1200	360	340	1830
28 N	1680	610	1070	280	260	1670
	1560	600	960	205	190	1550
M	1450	580	870	130	110	1450
	1340	560	780	70	42	1350
29 N	1250	540	710	50	28	1250
	1150	530	620	40	22	1160
M	1060	510	550	30	18	1070
	980	490	490	20	12	980
30 N	890	460	430	15	8	890
	800	440	360	10	7	810
M	700	410	290	5	3	700
	610	390	220	4	2	600
1 N	540	370	134	3	2	530
	460	350	110	2	1	460
M	400	330	70			400
	350	310	40			350
	300	300				300

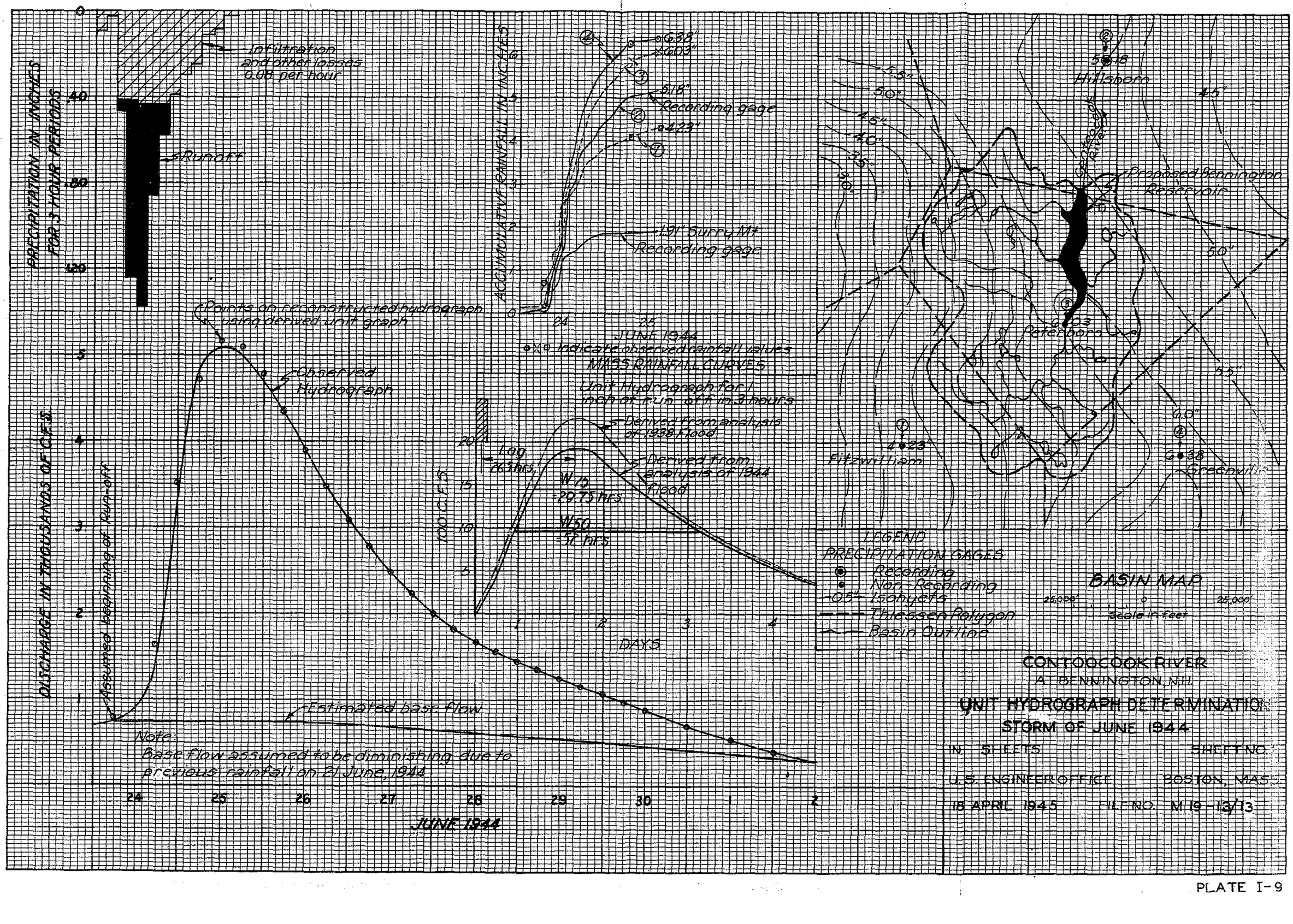

PRECIPITATION IN INCHES FOR 3 HOUR PERIODS
DISCHARGE IN THOUSANDS OF CFS
Assumed beginning of runoff
ACCUMULATIVE RAINFALL IN INCHES
Infiltration and other losses 0.06 per hour
Runoff
Points on reconstructed hydrograph using derived unit graph
Observed Hydrograph
Estimated base flow
Note:
Base flow assumed to be diminishing due to previous rainfall on 21 June, 1944
24 25 26 27 28 29 30 1 2
JUNE 1944
DAYS
.639"
.603"
5/16" Recording gage
.423"
1.91" Surry Mt Recording gage
Indicate observed rainfall values
MASS RAINFALL CURVES
Unit Hydrograph for 1 inch of run off in 3 hours
Derived from analysis of 1938 flood
Derived from analysis of 1944 flood
Log 26 hrs
W 12 20.75 hrs
W 50 52 hrs
Hillsboro
Proposed Bennington Reservoir
Peterboro
Fitzwilliam
Greenville
LEGEND
PRECIPITATION GAGES
Recording
Non-Recording
Isohyets
Thiessen Polygon
Basin Outline
BASIN MAP
Scale in Feet
CONTOOCOOK RIVER
AT BENNINGTON, N.H.
UNIT HYDROGRAPH DETERMINATION
STORM OF JUNE 1944
IN SHEETS SHEET NO.
U.S. ENGINEER OFFICE BOSTON, MASS.
18 APRIL 1945 FILE NO. M 19-13/13

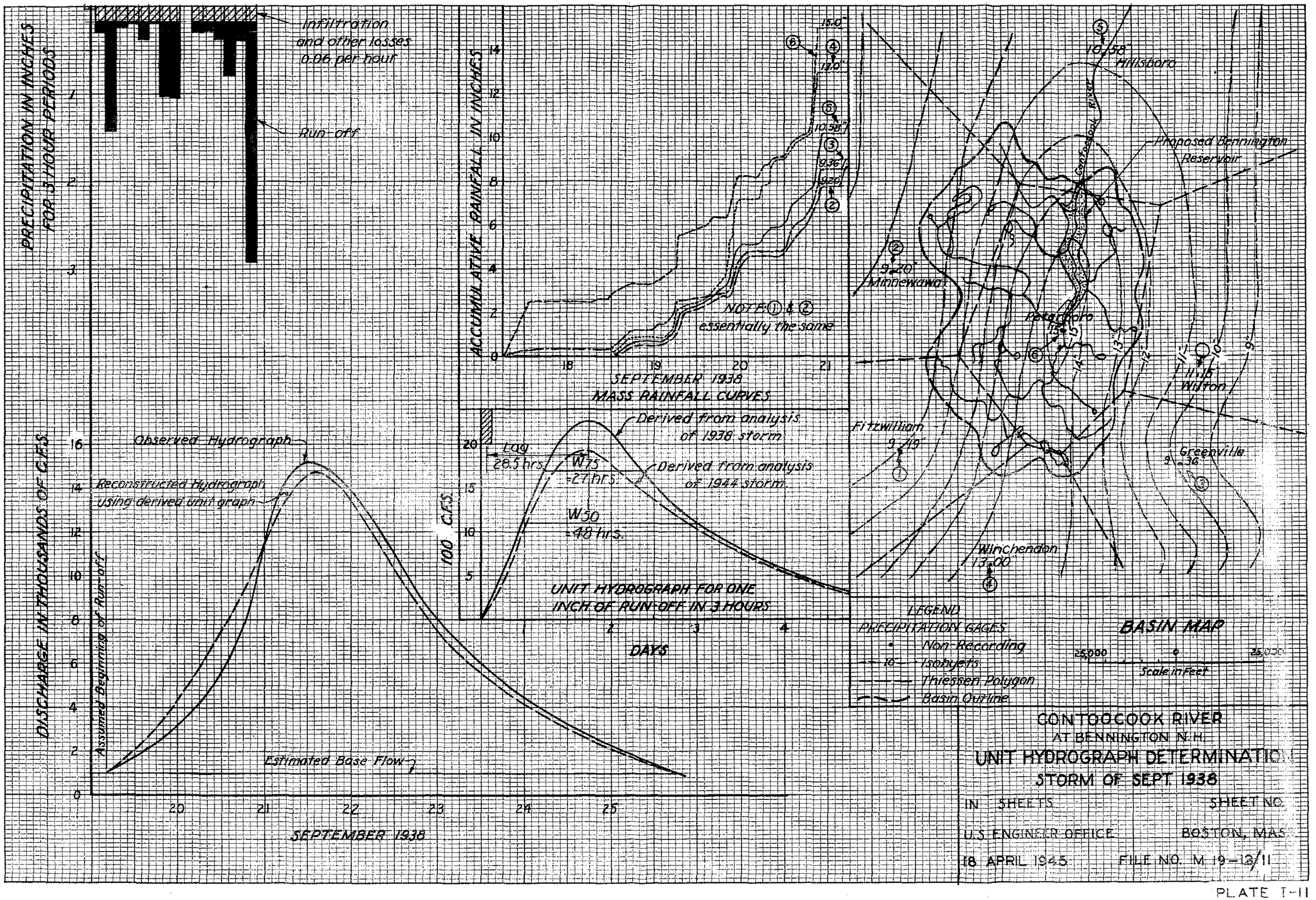
PLATE I-9

Computed by B.L.P.
Date Mar. 1945

UNIT HYDROGRAPH DETERMINATION

STREAM Contoocook River LOCATION Bennington, N. H.
DRAINAGE AREA 186 SQ. MI.
STORM OF Sept. 1938 PREPARED BY Boston DIST. N.E. DIV.
AV. RAINFALL 9.91 IN.; RAINFALL-EXCESS 7.40 IN.; F_{av}, 0.06 IN./HR.
L 20 mi; L_{co} 7.70 mi; $(LL_{co})^{0.3}$ 4.53; t_R 3 hrs;
LAG(t_{pR}) 28.5 hrs; C_{tR} 6.30; q_{pR} 12.2 c.f.s./sq.mi.; C_p 640 347
LAG(t_{cmR}) 42.2 hrs; W_{50} 47.0 hrs; W_{75} 26.4 hrs; SLOPE

TIME	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED 6 HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED 3 HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
1910A						
4P	1000	1000	0	0	0	1000
10P	1550	1000	550	868	1175	1800
4A	2200		1200	1940	2130	2750
2010A	2900		1900	2195	2130	3900
4P	3850		2850	1675	1530	5300
10P	5100		4100	1200	1100	6900
4A	6900		5900	870	800	8650
2110A	10000		9000	630	570	10700
4P	13750		12750	440	400	12900
10P	15000		14000	285	260	14300
4A	15000		14000	210	190	14650
2210A	14200		13200	140	120	13850
4P	13500		12500	85	70	12450
10P	12050		11050	45	40	10850
4A	10500		9500	15	10	9300
2310A	8850		7850	8	6	8350
4P	7650		6650	6	6	7100
10P	6600		5600	4	2	6100
4A	5700		4700	2	2	5250
2410A	4900		3900	1		4600
4P	4250		3250			3950
10P	3600		2600			3400
4A	3050		2050			2850
2510A	2550		1550			2350
4P	2100		1100			1950
10P	1700		700			1550
4A	1300		300			1250



CONTOOCOOK RIVER
AT BENNINGTON N.H.
UNIT HYDROGRAPH DETERMINATIO
STORM OF SEPT. 1938

IN SHEETS SHEET NO.

U.S. ENGINEER OFFICE BOSTON, MASS.

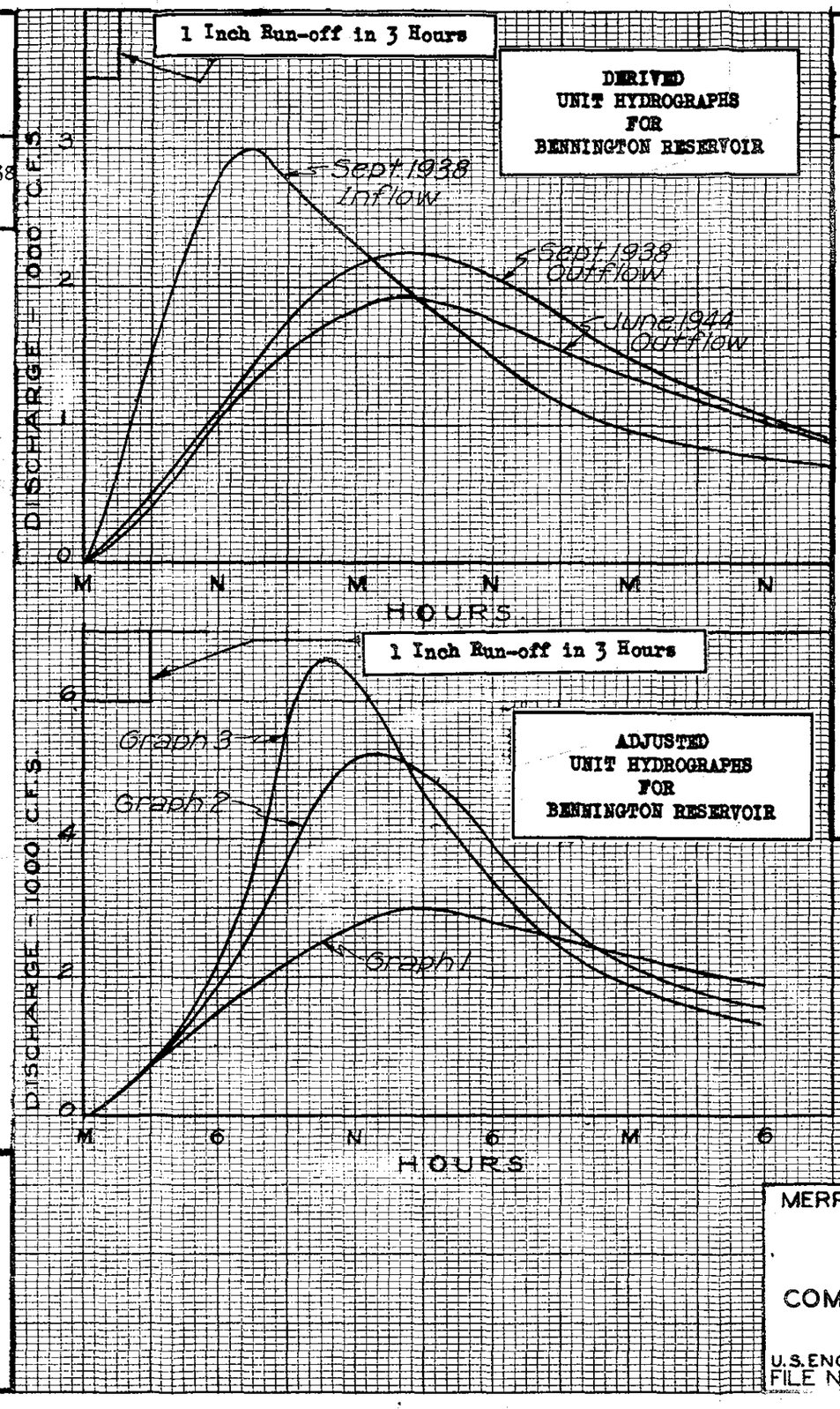
18 APRIL 1945 FILE NO. M 19-12/11

PLATE I-11

ANALYSIS OF UNIT HYDROGRAPH

SYMBOL	EXPLANATION	UNIT	JUNE 1944 OUTFLOW	SEPT.1938 OUTFLOW	SEPT.1938 INFLOW
D.A.	Drainage Area	Sq.Mi.	186	186	186
	Storm of		June '44	Sept. '38	Sept.'38
$R_{(ave)}$	Total Weighted Basin RF	In.	5.80	9.91	9.91
Re	Rainfall Excess	In.	2.38	7.40	7.40
$F_{(ave)}$	Infiltration	In./hr	0.082	.056	.056
Q_{max}	Peak Discharge	c.f.s.	5,100	15,200	18,000
q	Peak Discharge per Sq.Mi.	c.s.m.	27.4	81.6	81.6
L	Length of Drainage Basin	Miles	20.0	20.0	15.4
$L_{(c.a.)}$	Length from center of gravity to point of discharge	Miles	7.70	7.70	7.70
$(LL_{ca})0.3$			4.53	4.53	4.20
t_R	Unit Rainfall Duration	Hours	3	3	3
$Lag_{(t_pR)}$	Time of Peak Concentration	Hours	26.1	28.5	13.3
C_{tR}	Drainage Basin Coefficient		5.78	6.30	3.10
q_{pR}	Peak Discharge of Unit Hydrograph	c.s.m.	10.3	12.2	16.1
C_p640	Drainage Basin Coefficient		268	347	214
$Lag(t_cmR)$	Time of Volume Concentration	Hours	48.9	42.2	38.1
w_{50}	Width at 50% of Peak Discharge	Hours	51.9	47.0	30.0
w_{75}	Width at 75% of Peak Discharge	Hours	29.2	26.4	15.8

For detailed explanation of use of above symbols refer to "Synthetic Unit Graphs" by Franklin F. Snyder, Transactions American Geophysical Union, 1938.



BENNINGTON RESERVOIR

UNIT HYDROGRAPH VALUES

TIME Hour	GRAPH L c.f.s.	GRAPH 2 c.f.s.	GRAPH 3 c.f.s.
M			
1.5	300	300	300
3	650	700	700
4.5	1190	1200	1300
6	1500	1900	2100
7.5	1880	2650	3400
9	2250	3650	5700
10.5	2500	4740	6600
N	2800	5220	6250
1.5	2950	5200	5530
3	3000	4980	4630
4.5	2900	4500	3950
6	2750	3950	3370
7.5	2600	3350	2850
9	2500	2830	2450
10.5	2400	2470	2120
M	2300	2180	1850
1.5	2200	2000	1670
3	2100	1830	1550
4.5	2000	1720	1440
6	1900	1610	1360
7.5	1800	1540	1250
9	1700	1490	1200
10.5	1600	1400	1150
N	1500	1350	1100
1.5	1400	1300	1000
3	1320	1200	990

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

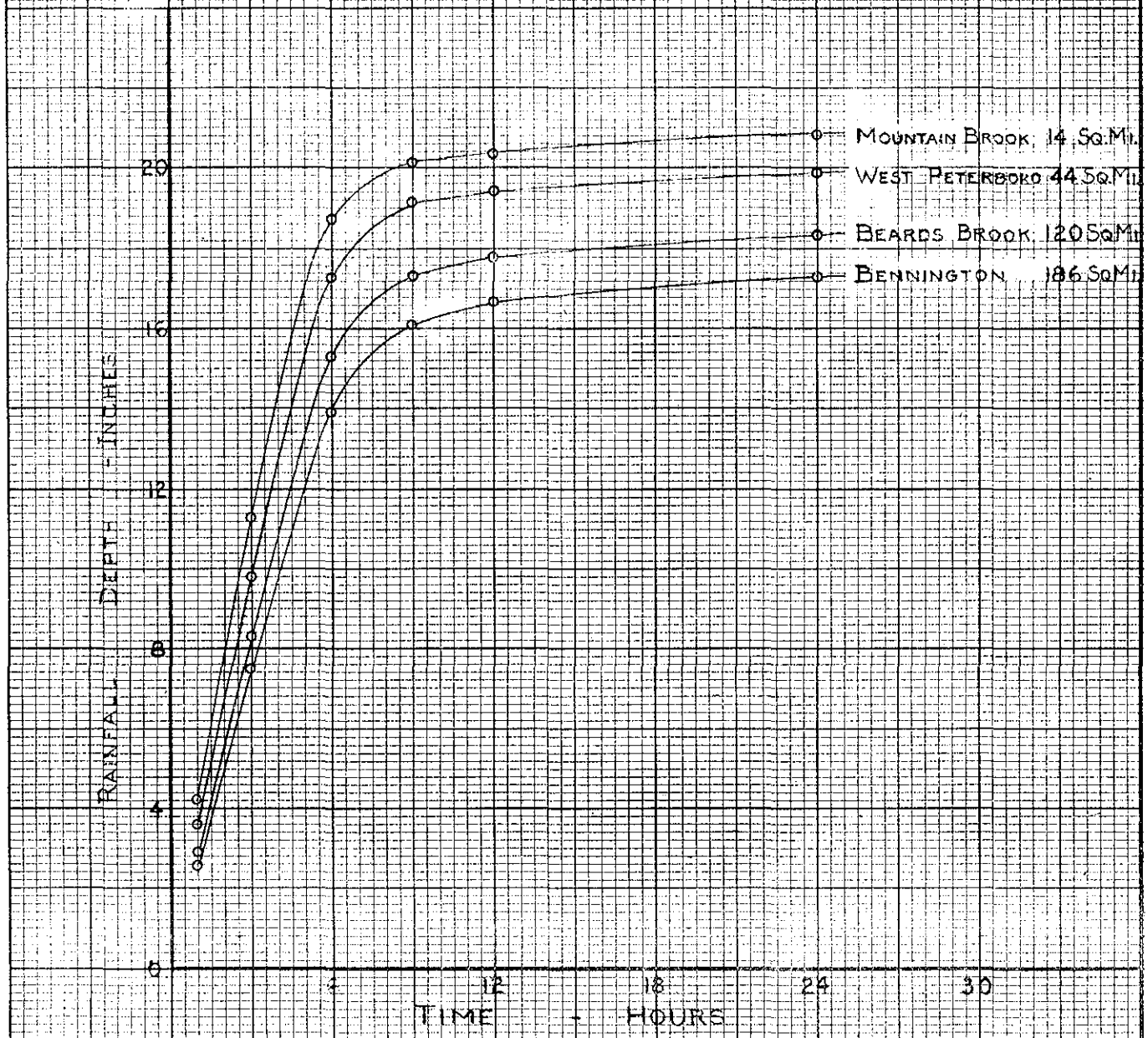
COMPARISON OF UNIT GRAPHS

U.S.ENGINEER OFFICE
FILE NO. M 19-13/14

BOSTON, MASS.
18 APRIL 1945

PLATE I-12

THESE CURVES ARE BASED ON DATA SUPPLIED BY THE HYDROMETEOROLOGICAL SECTION OF THE WEATHER BUREAU. REFER TO LETTER FROM CHIEF OF ENGINEERS, 8 DECEMBER 1944



MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK RIVER BASIN

CURVES OF LIMITING RAINFALL

U.S. ENGINEER OFFICE
FILE NO. 100-50/5

BOSTON, MASS.
18 APRIL 1945

PLATE I-13

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

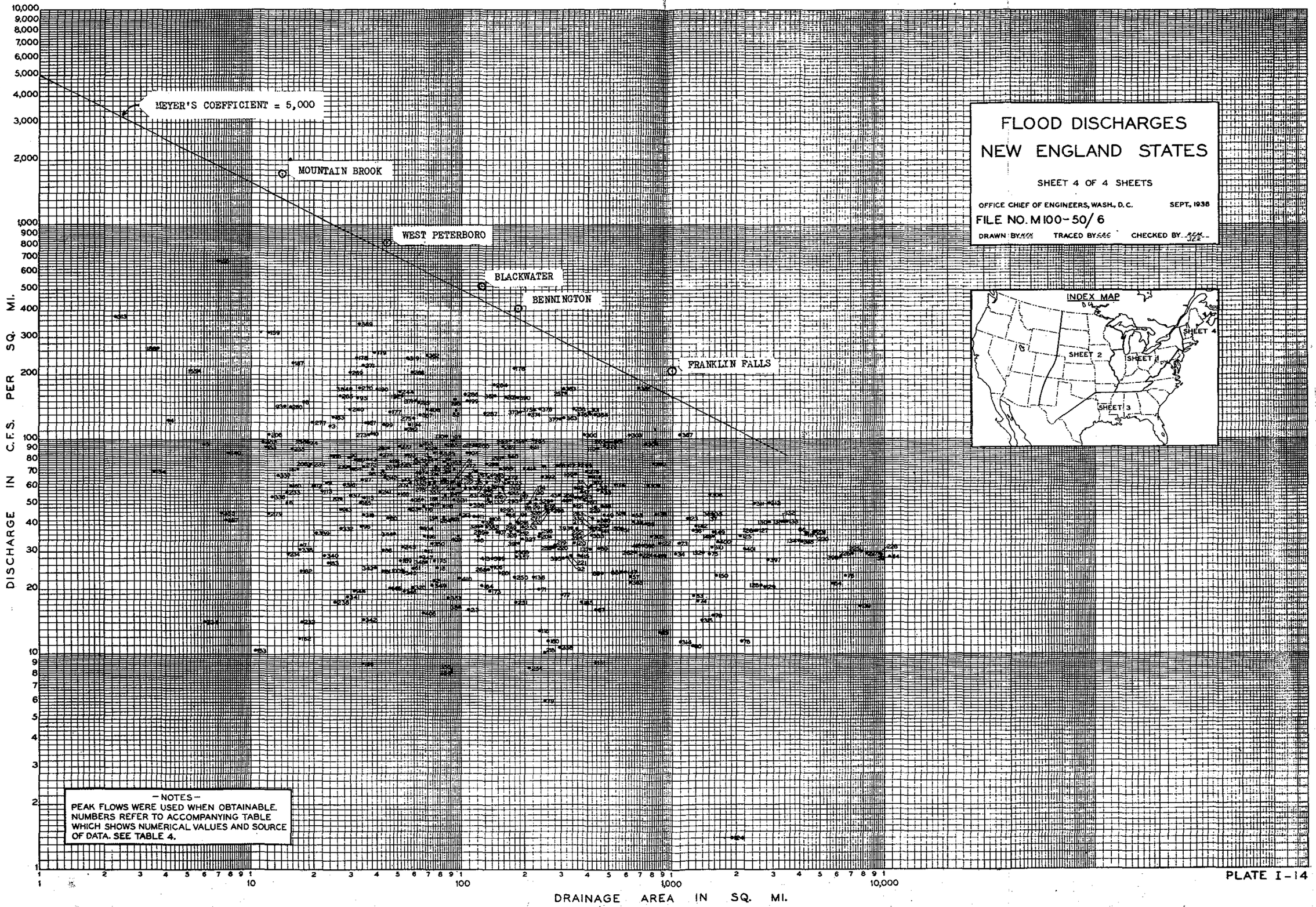
FLOOD DISCHARGES
NEW ENGLAND STATES

SHEET 4 OF 4 SHEETS

OFFICE CHIEF OF ENGINEERS, WASH., D.C. SEPT., 1938

FILE NO. M100-50/6

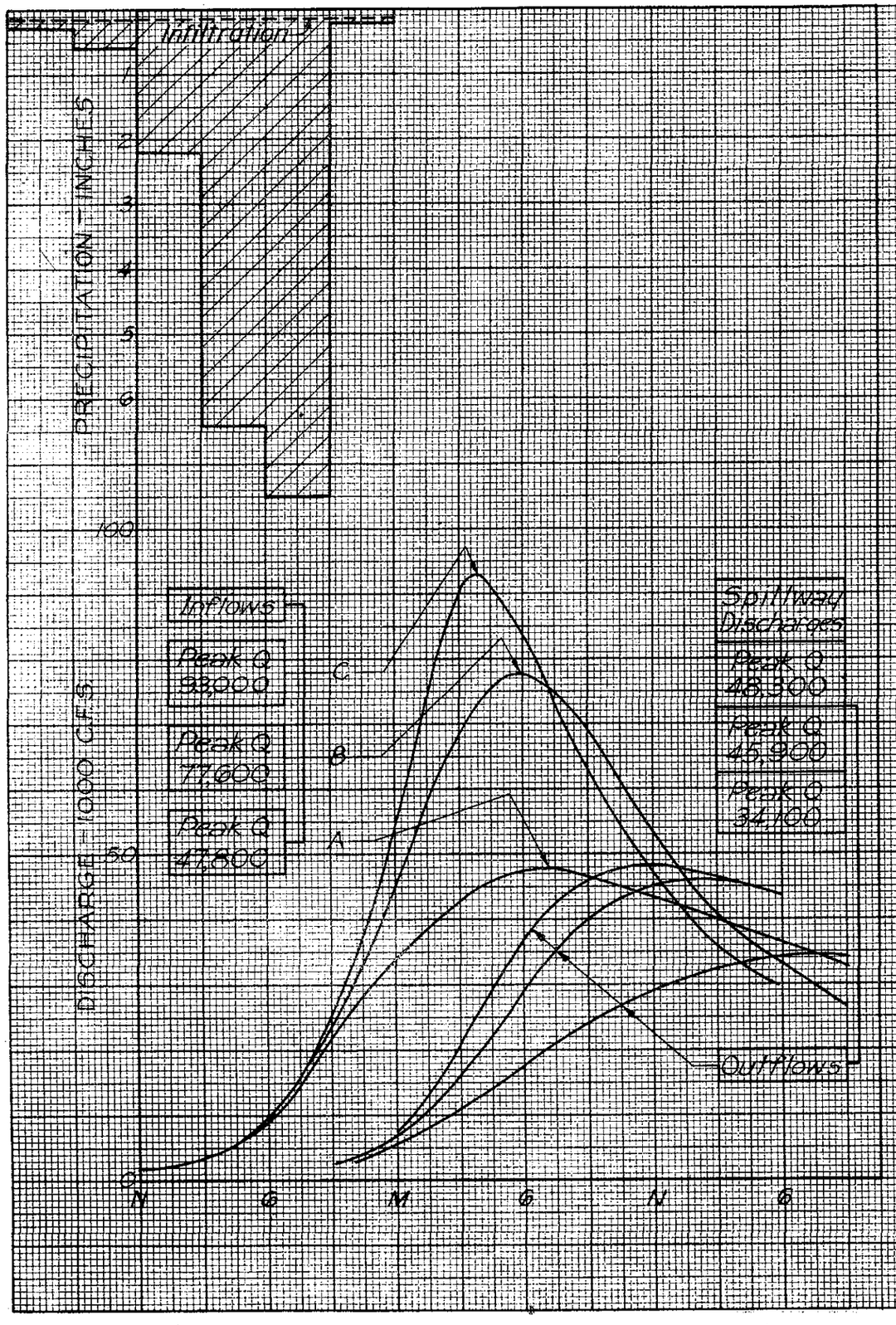
DRAWN BY: [illegible] TRACED BY: [illegible] CHECKED BY: RFM JCR

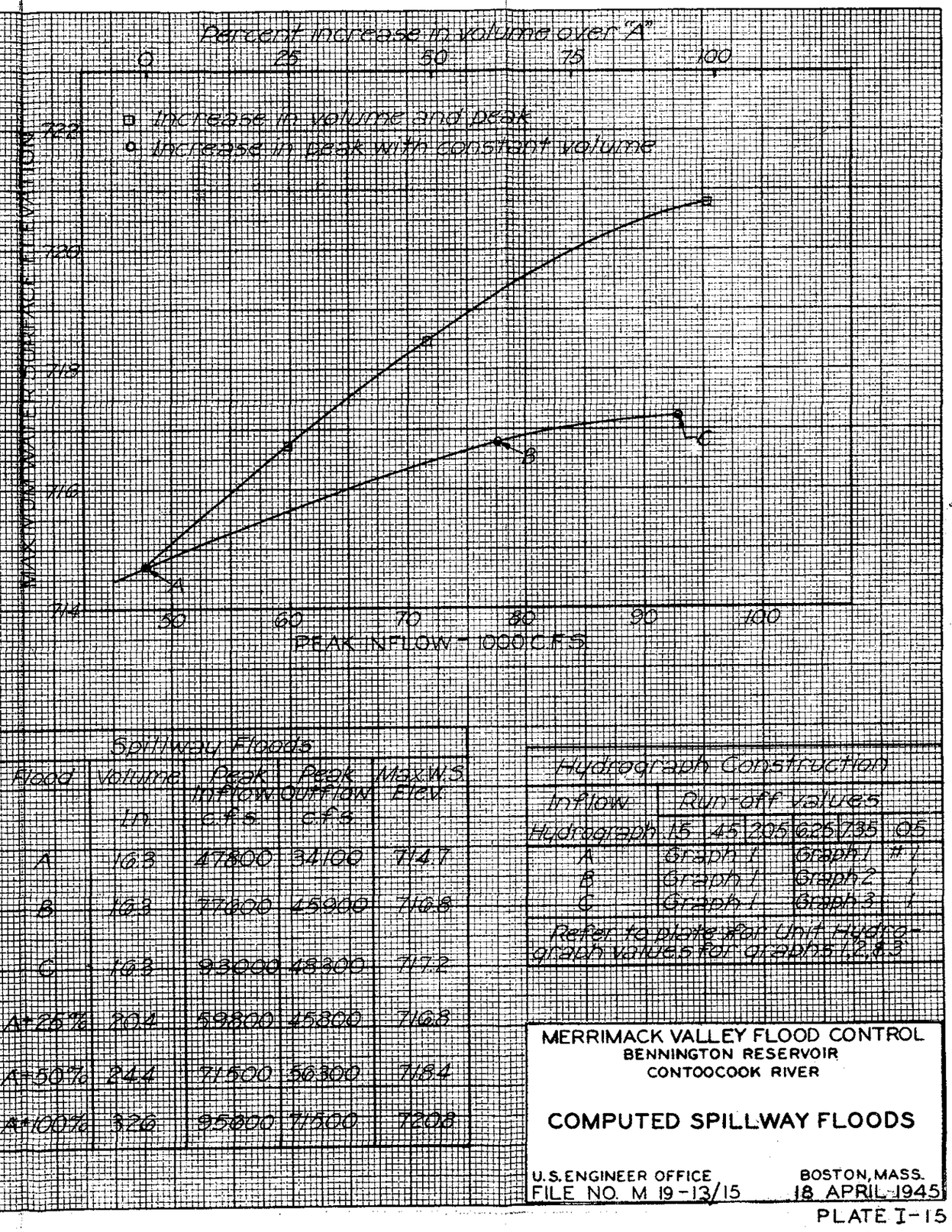


— NOTES —

PEAK FLOWS WERE USED WHEN OBTAINABLE. NUMBERS REFER TO ACCOMPANYING TABLE WHICH SHOWS NUMERICAL VALUES AND SOURCE OF DATA. SEE TABLE 4.

PLATE I-14





Spillway Floods

Flood	Volume In.	Peak Inflow C.F.S.	Peak Outflow C.F.S.	Max. W.S. Elev.
A	16.3	47800	34100	714.7
B	16.3	77600	45900	716.8
C	16.3	93000	48300	717.2
A+25%	20.4	59800	45300	716.8
A+50%	24.4	71500	56300	718.4
A+100%	32.6	95800	71500	720.8

Hydrograph Construction

Inflow Hydrograph	Run-off values .15	.45	.205	.626	.735	.05
A	Graph 1			Graph 1		#1
B	Graph 1			Graph 2		1
C	Graph 1			Graph 3		1

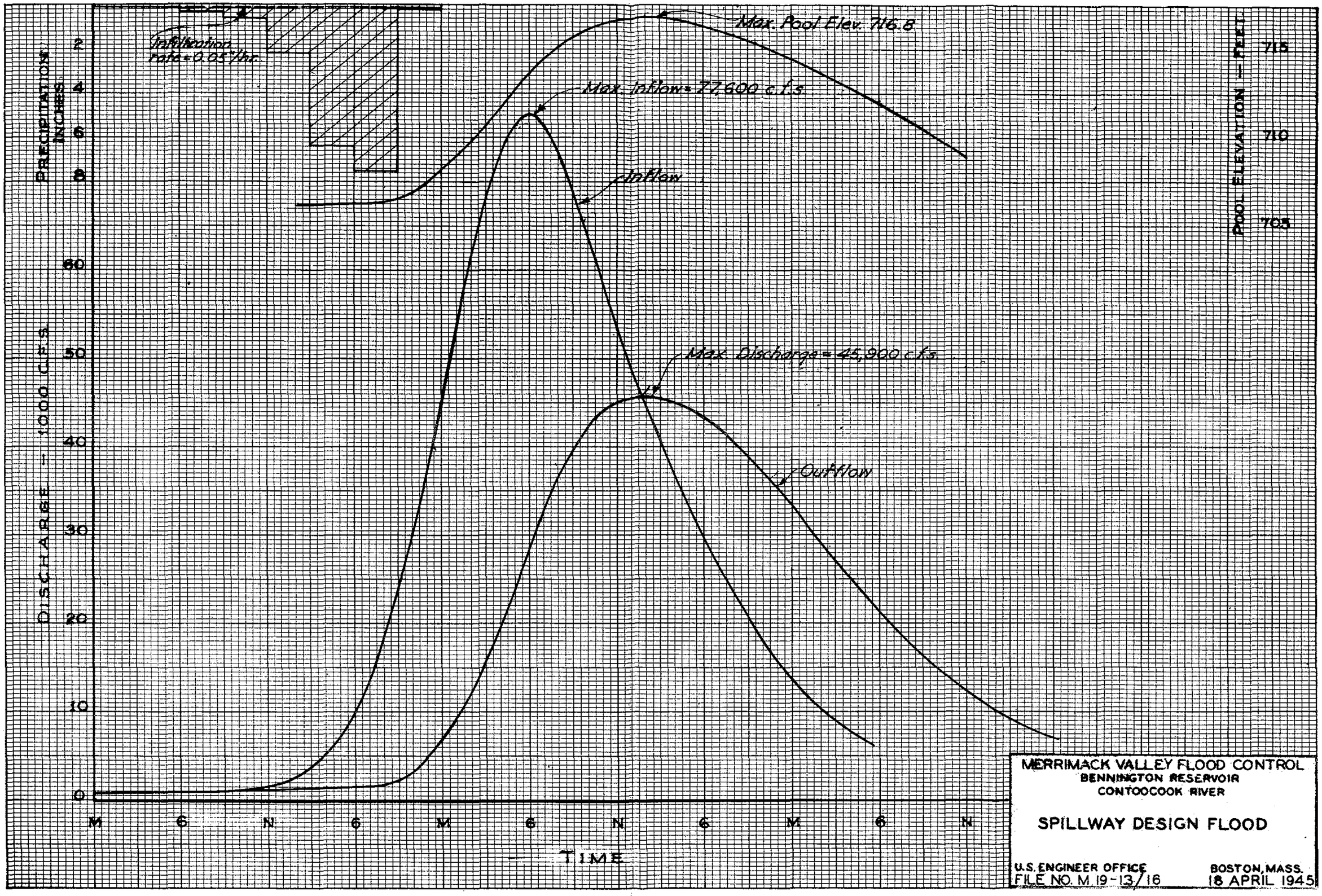
Refer to plate #1 for Unit Hydrograph values for graphs 1, 2, & 3

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

COMPUTED SPILLWAY FLOODS

U.S. ENGINEER OFFICE — BOSTON, MASS.
FILE NO. M 19-13/15 — 18 APRIL 1945

PLATE I-15



MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

SPILLWAY DESIGN FLOOD

U.S. ENGINEER OFFICE
FILE NO. M.19-13/16

BOSTON, MASS.
18 APRIL 1945

PLATE I-16

EFFECT OF MOUNTAIN BROOK RES. ON BENNINGTON RES. SPILLWAY DESIGN FLOOD

Natural inflow to M.B. Res.

Case 1- Natural inflow to Ben. Res. Case 2- Modified outflow from M.B. Res.

Modified inflow to Ben. Res.

Discharge in 1000 c.f.s.

Time in hours

EFFECT OF WEST PETERBORO RES. ON BENNINGTON RES. SPILLWAY DESIGN FLOOD

Natural inflow to W.P. Res.

Natural inflow to Ben. Res.

Case 1- Modified outflow from W.P. Res. Case 2- Modified inflow to Ben. Res.

Discharge in 1000 c.f.s.

Time in hours

Bennington Reservoir Spillway Design Flood

Reservoir Inflow without M.B. & W.P.

Reservoir Inflow with M.B. & W.P.

Reservoir Outflow without M.B. & W.P.

Reservoir Outflow with M.B. & W.P.

Max. Q = 45,900 c.f.s.
Max. El = 716.8 Ft.

Max. Q = 42,600 c.f.s.
Max. El = 716.2 Ft.

Discharge in 1000 c.f.s.

Time in hours

Notes:

Natural inflows to Mountain Brook and West Peterboro Reservoirs are based on maximum possible precipitation values for 180 sq. mi. (see Plate I-13) using Unit Graphs for Mountain Brook and West Peterboro drainage areas.

Decrease in Bennington spillway design flood due to either Mountain Brook or West Peterboro Reservoirs.

Increase in Bennington spillway design flood due to either Mountain Brook or West Peterboro Reservoirs.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

EFFECT OF UPSTREAM RESERVOIRS ON SPILLWAY DESIGN FLOOD

U.S. ENGINEER OFFICE
FILE NO. M 19-13/17

BOSTON, MASS.
18 APRIL 1945

PLATE I-17

Max water surface Elev. 706.0

Spillway Crest

Pool Elevation

Bennington inflow reduced by West Peterboro

DISCHARGE 1000 C.F.S.

RESERVOIR STAGE

1 Gate Open

2 Gates Open

3 Gates Open

4 Gates Open

2 Gates Open

2 Gates & Spillway

3 Gates & Spillway

4 Gates Open

5 Gates Open

6 Gates Open

Reservoir Outflow

MARCH 1936 APRIL

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

EFFECT ON 1936 FLOOD INITIAL DEVELOPMENT

U.S. ENGINEER OFFICE BOSTON, MASS.
FILE NO. M 19-13/18 18 APRIL 1945

PLATE I-18

Computed inflow to Bennington Reservoir allowing for Valley Storage

Initial Spillway Crest El. 705

Max reservoir stage El. 702.1

Reservoir stage

Inflow reduced by West Peterboro Reservoir

Flood occurring at Bennington, N.H.

1 gate open

2 gates open

4 gates open

5 gates open

6 gates open

Reservoir discharge

Discharge in 1000 C.F.S.

Reservoir Stage in Feet above M.S.L.

September & October 1938

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

EFFECT ON 1938 FLOOD
INITIAL DEVELOPMENT

U.S. ENGINEER OFFICE — BOSTON, MASS.
FILE NO. M 19-13/19 — 18 APRIL 1945

PLATE I-19

Max. water surface Elev. 712.9

Spillway Crest

Pool Elevation

Bennington inflow reduced by West Peterboro

Variable Operation up to 4 Gates as required.

2 Gates Open

2 Gates & Spillway

4 Gates Open

Conservation Pool

Reservoir Outflow

RESERVOIR STAGE

712
710
708
706
704
702
700

FLOW-IN THOUSANDS OF C.F.S.

14
12
10
8
6
4
2
0

12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 1 2 3 4 5 6 7 8

MARCH

APRIL

1936

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

EFFECT ON 1936 FLOOD ULTIMATE DEVELOPMENT

U.S. ENGINEER OFFICE BOSTON, MASS.
FILE NO. M 19-13/20 18 APRIL 1945

PLATE I-20

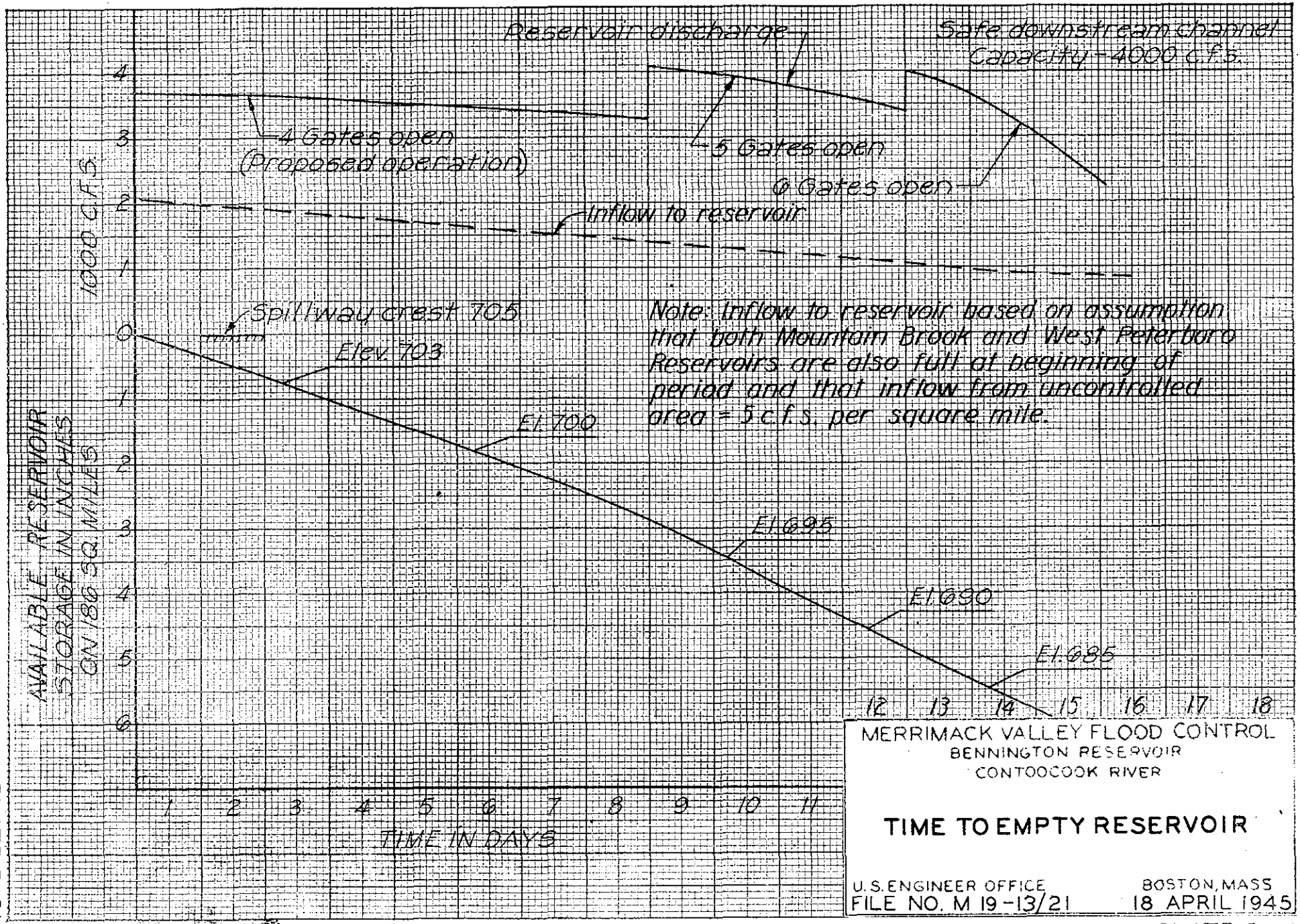


PLATE I-21

PLATE I-21

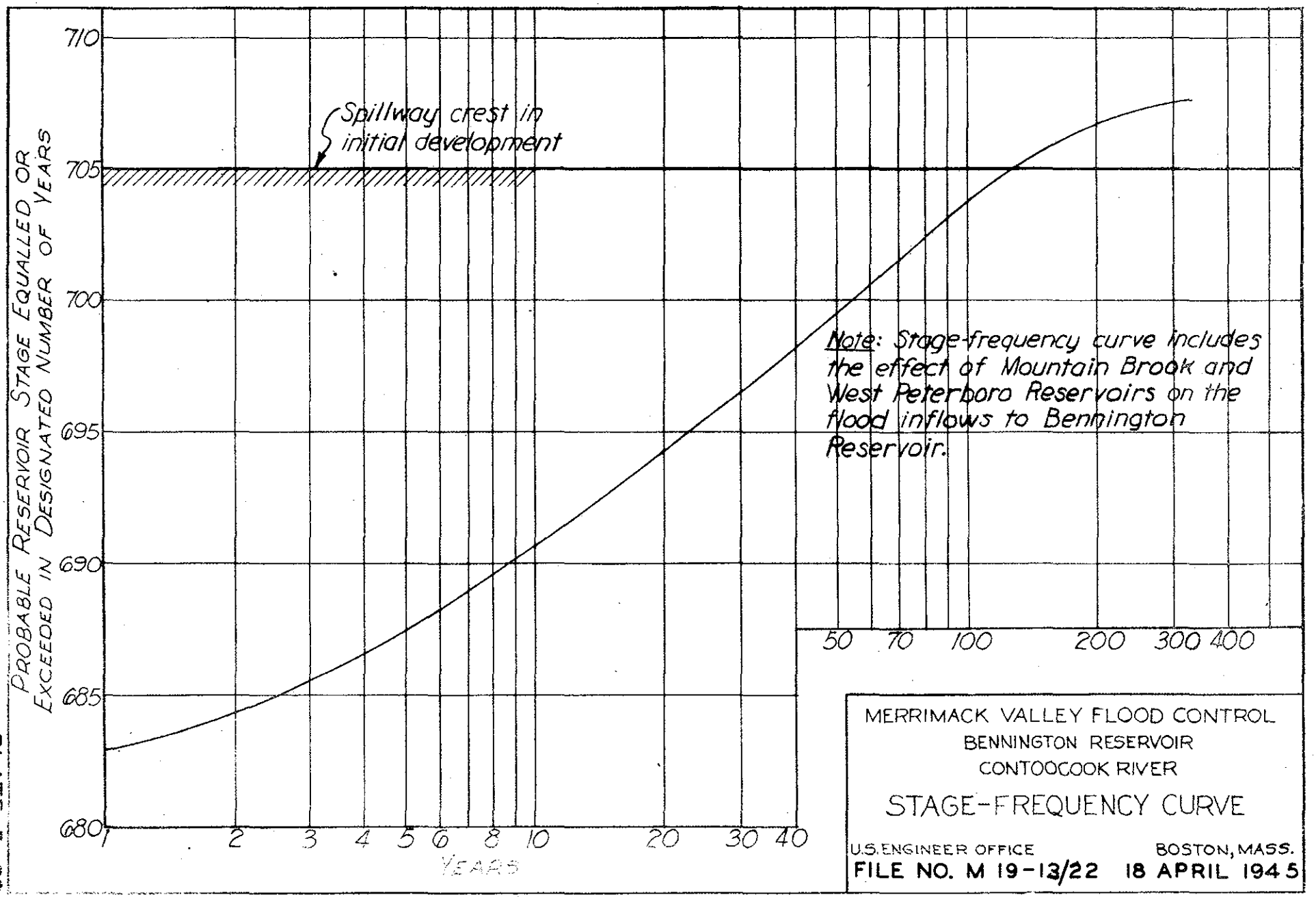


PLATE I-22

PLATE I-22

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX II

EXPLORATION, GEOLOGY, CONSTRUCTION MATERIALS, SOIL DATA AND ANALYSIS

To accompany definite project report
Dated April 1945

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX II.

EXPLORATION, GEOLOGY, CONSTRUCTION MATERIALS, SOIL DATA AND ANALYSIS

C O N T E N T S

Paragraph	Title	Page
a.	Exploration	II-1
b.	Geology	II-3
c.	Construction Materials	II-6
d.	Soil Data	II-12
e.	Embankment Design	II-14
f.	Spillway and Outlet Structure Foundation	II-18
g.	Approach and Discharge Channels	II-20
h.	Upstream Cofferdam Design	II-21

PLATES

Plate	Title
II-1	Plan of Foundation Exploration
II-2	Geological Profiles - No. 1
II-3	Geological Profiles - No. 2
II-4	Record of Foundation Exploration No. 1
II-5	Record of Foundation Exploration No. 2
II-6	Record of Foundation Exploration No. 3
II-7	Record of Foundation Exploration No. 4
II-8	Record of Foundation Exlporation No. 5
II-9	Plan of Borrow Areas
II-10	Plan of Borrow Exploration
II-11	Soil Data Summary
II-12	Seepage analysis
II-13	Stability and Seepage Analysis
II-14	Filter Analysis
II-15	Stability Analysis of Upstream Cofferdam

DEFINITE PROJECT REPORT
BENNINGTON RESERVOIR

APPENDIX II

EXPLORATION, GEOLOGY, CONSTRUCTION MATERIALS, SOIL DATA AND ANALYSIS

a. Exploration.- (1) Reconnaissance.- Reconnaissance was conducted in the general vicinity selected for the project location. Topography and geography were studied and surficial examination was made to investigate the principal geological formations. Observations indicated that a satisfactory foundation could be developed and that suitable construction materials were available within reasonable hauling distance.

(2) Seismic Exploration.- Under the supervision of Mr. E. R. Shepard, Office of the Chief of Engineers, seismic explorations were conducted at 18 locations within the project area (Plate II-1). The explorations were made primarily to determine depth to bedrock. A table of information obtained from the investigation is shown on Plate II-8. Some indication of the general character of the overburden was obtained by wave velocities above the rock surface.

(3) Subsurface Exploration.- (a) Foundation Exploration.- Foundation conditions at the dam site were investigated by 67 drill holes, 6 shallow test pits and 2 deep, sheeted test pits. The location of explorations are shown on Plate II-1 and the logs of explorations are included on Plates II-4 to II-8, inclusive. The drill holes were advanced using a three-inch casing; samples of overburden were obtained with a 2-inch drive sampler, and rock cores were obtained with the use of a 1-5/8 inch diamond bit. The two deep test pits in the spillway area T45p and T60p, were excavated to depths of 10 and 24 feet, respectively. Samples of each stratum encountered in drill holes and test pits were obtained for classification, and undisturbed samples of principal materials were obtained from test pits for determination of natural soil properties.

(b) Borrow Exploration.- Borrow materials have been investigated by twenty drill holes and 52 shallow test pits. The locations of explorations are shown on Plate II-9 and the logs of selected explorations are included on Plate II-10. The drill holes were advanced using a 2-inch casing and samples of overburden were obtained with a one-inch drive sampler. No rock coring was done for borrow exploration. All drill holes were located in or near the impervious borrow area with the object of determining the extent of material available. Three test pits in the impervious borrow area, six pits in the pervious borrow area, and forty-three pits scattered within a radius of three miles of the dam site were explored during the investigation. All pits were sampled for classification of the principal materials. Field tests for natural density were performed in the

pits located in the pervious borrow area and undisturbed samples were taken from the pits in the impervious borrow area for laboratory testing.

(4) Observation Wells.- Eight observation wells have been installed in drill holes within the dam site area to obtain detailed knowledge of ground water conditions (Plate II-1). Each observation well consists of a 1-1/4 inch well point and a riser pipe that extends slightly above ground surface. Sand and gravel has been placed around the well points to prevent their being plugged by movement of fine material. Each well point has been placed in the most pervious zone encountered in the drilling of the hole. Five of the wells are located on the western side of the river and have their points exposed to the pervious layer that underlies the till blanket. The remaining two wells are in the spillway area and have been embedded in the weathered granite that underlies the till. Both wells in the spillway area have hydrostatic heads at or above the ground surface. Except for the two wells in the spillway area which were not installed until January 1945, all the wells were read daily for nearly two months. Since daily fluctuations were not appreciable, readings have been continued at intervals of approximately one week.

Observations to date are summarized in the following tabulation:

Observation Well	Ground Surface Elevation	Elevation of Water in O.W.	
		Minimum	Maximum
D 8	695.1	685.4	689.1
11	671.7	673.4	677.1
18	671.9	673.1	676.5
26	667.7	666.5	669.0
49	681.6	676.8	679.7
57	700.3	689.1	693.2
66	676.7	673.0	675.9
67	673.4	674.1	675.3

(5) Pressure Tests in Bedrock.- Two drill holes in the spillway area, D66 and D67, indicated the presence of a zone of badly weathered rock immediately beneath the till body. D66 penetrated a total of 40 ft. of rock and D67 penetrated 47 feet of rock. In each of these holes bedrock was tested for its total leakage and then 5 feet segments of the rock were tested to determine the zones of greatest leakage. Each segment was isolated and subjected to a flow of water under a pressure of approximately 70 lbs. per sq. inch. D66 had approximately 10 feet of weathered rock that was too soft to allow testing. The remainder of the rock was relatively tight except for a large seam near the bottom of the hole that accommodated a flow of ten gallons per minute. D67 had 20 feet of decomposed rock in which the pressure testing equipment could not be sealed, ten feet of moderately fractured rock with a total loss of 5 gallons per minute

under 70 lbs. per sq. in. pressure, and 17 feet of fresh unfractured rock. Pressure applied to the water at D67 caused the water level at D66 to rise, thus indicating a network of interconnected fractures within a localized area.

(6.) Pumping Tests.- Small scale pumping tests were conducted at D66 and D67 to determine the amount of water available for percolation within the zone of decomposed rock, and to determine the feasibility of removing the excessive hydrostatic head during construction of the spillway. A four hour test at D67 gave an average flow of 3.5 gallons per minute and lowered the water level in D66 a distance of 16 inches. Sixteen hours after the cessation of pumping, the water level in D66 had risen only 9 inches. Two days later when the water level had returned to normal, pumping was begun at D66 and observations taken at D67. Four hours of pumping lowered the water level in D67 a distance of 20 inches. One hour after the pumping ceased, the water level had risen 5 inches. The results indicate a limited supply of water percolating through numerous interconnected fractures.

b. Geology.- (1) Regional.- (a) Contoocook Valley.- The rocks of the Contoocook Valley are metamorphic and igneous of Paleozoic age. The metamorphic rocks are schists and gneisses which were deposited originally as flat beds below the water surface. A period of diastrophism which occurred subsequent to this deposition resulted in the crumbling and folding of the layers until they appeared at every attitude. Large amounts of magma were injected into the metamorphosed rocks, where relatively slow cooling occurred and granite or related granitic rocks formed. A period of regional uplift followed and streams began the gradual erosion of the overlying metamorphic rocks. As the drainage pattern developed, the streams cut downward through the metamorphic rocks and exposed large areas of the granite. The granite was not a durable rock and has since been affected by weathering to depths of as much as 50 feet. At the beginning of the Pleistocene period, the Contoocook Valley had been developed, and the river was flowing northward in a bedrock channel. During the Pleistocene period, the valley was covered by a portion of the continental glacier that occupied Canada and the Northern part of the United States. The glacier modified existing topography by erosion and deposition. Erosion by the glacier consisted of bevelling the northern slopes of hills and steepening their southern slopes. In addition, it scoured and widened north-south valleys and tended to deepen and widen saddles transverse to its line of motion. Deposition by the glacier caused the greatest change in topography. Glacial deposits are of two types: the unsorted, unstratified glacial till deposited directly from the ice, and the sorted gravels, sands, silts and clays that are washed from the ice and deposited in water. Glacial till may consist of a

heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders, or some of these sizes may be lacking. In general, the till of the Contoocook Valley does not have material as fine as the clay sizes in any appreciable amount. Most of the till may be broken down into two classes: sandy till, which has a predominance of sand sizes, and silty till which has a predominance of silt sizes. Glacial till is highly stable, capable of furnishing good support and it practically impervious. It was deposited in elliptical shaped hills known as drumlins, and as blankets which were smeared across the valleys, partially filling them and disorganizing the drainage system. In many cases interstream divides were buried by glacial deposits and streams now pass from one old drainage basin to another. Where streams still flow within the old valley, they are seldom in the old channel, therefore, when a stream has a rock floor, there may be a buried channel nearby. The sorted materials consist of gravels, sands, silts and clays or, as in the case of a modified glacial drift, combinations of these sizes. These materials have been washed from the ice, sorted or partially sorted and deposited at some point where the velocity of the water slackens. The Contoocook Valley has many of these materials in the form of terrace deposits, eskers, and delta deposits. An extensive blanket of sediments was deposited on the floor of the valley during the recession of the glacier when the northward flowing Contoocook River was dammed by the ice front and a large glacial lake formed. When the ice front retreated northward the lake was drained into the Merrimack River Valley and erosion of the present Contoocook channel began.

(b) Vicinity of Dam Site.- In the vicinity of the Bennington dam site the river flows through extensive shallow surface deposits of unconsolidated sediments which overlie deposits of glacial till in some areas as described in the following paragraphs. Bedrock is deeply buried throughout the vicinity except where it outcrops in the floor of the river approximately 3/4 mile downstream from the dam. The western wall of the present valley is composed of a valley terrace deposit and the eastern wall is composed of glacial till. Esker and kame deposits are common throughout the area.

(c) Source of Data.- Information pertaining to historical and existing geology of the Contoocook Valley has been abstracted from reports on the Bennington site presented to the Boston District Office by Mr. Sidney Paige, Geologist, North Atlantic Division Office of the U. S. Engineers, by Mr. Charles P. Berky, Consulting Geologist, Columbia University, and by Mr. Irving B. Crosby, Consulting Geologist, Boston.

(2) Dam Site.- (a) General.- The geology of the dam site is discussed in the following sub-paragraphs according to principal conditions encountered. A plan of foundation exploration showing location of drill holes, seismic lines and contours of till and bed-

rock is shown on Plate II-1. Geological profiles are shown on Plates II-2 and II-3, and graphic logs of individual explorations are shown on Plates II-4 through II-8. Plate II-8 includes a summary of information obtained from seismic investigation.

(b) East Abutment.- The east abutment of the dam and the embankment foundation to station 8 + 50 is composed of a compact glacial till that extends to bedrock at depths of approximately 50 feet. The till varies from sandy till near the surface to silty till which forms the major portion of the body. Overlying the till is a variable cover of silty sand and gravel. Cobbles and boulders occur occasionally in the till and frequently in the surface deposits. These conditions prevail within the embankment area and adjacent downstream area. Upstream from the embankment area the variable surface sand deposits increase in thickness.

(c) Spillway Area.- The spillway area extends from station 8 + 50 to station 11 + 50 on the centerline of the dam. The principal soil in this area is a compact sandy to silty till that extends from the bottom of the surface sediments, to bedrock at depths of 50 feet or more. The bedrock is a porphyritic granite and has a very badly weathered capping that is much more pervious than the overlying till. This weathered zone varies in depth from 5 to 30 feet or more and grades into fresh, moderately fractured granite. The ground water table is very close to the ground surface in this area and the hydrostatic head in the bedrock under the till is slightly above the ground surface. Conditions in the immediate upstream and downstream areas are essentially the same.

(d) River Channel Area.- The river channel area from station 11 + 50 to station 20 + 00 is composed of surface deposits averaging 20 feet in thickness, overlying an extensive body of sandy to silty till, that, in turn, overlies weathered granite at depths in excess of 50 feet. The surface of the till rises in both the upstream and downstream directions.

(e) Drainage Well Area.- The section of the embankment foundation from station 20 + 00 to 30 + 00 has 10 to 15 feet of unconsolidated surface sediments. Beneath these sediments, the body of sandy to silty till splits into two sections; The upper section tapers out laterally and disappears near station 30 + 00; the lower section dips sharply toward the west and disappears against the rock floor. The zone between the till blankets is occupied by sediments which range in permeability from pervious to semi-impervious. Bedrock in this area is approximately 100 feet below the surface. As shown on Profile B of Plate II-2, the upper section of till lenses out upstream permitting ready access of water to the underlying more pervious soils. Downstream from the dam the pervious substratum is cut off by contact of the two till layers.

(f) West Abutment.- The last 1000 feet of the embankment is underlain by large terrace deposits of sand and silt which extend from the ground surface to bedrock at depths which range from 100 to 150 feet. Materials in this terrace grade from loose medium sand at the surface to moderately compact silt at considerable depth.

(3) Borrow Sources.- In the vicinity of the dam site, the Contoocook Valley has large amounts of glacio-fluvial material in the form of eskers, kames and terrace deposits. These deposits vary in the amount of sorting they have received, hence variations in grading and degree of permeability are available. Also available in the immediate vicinity of the site are massive deposits of glacial till which are in general overlain by a variable thickness of sediments. The glacial tills in the region are described as sandy to silty tills, the former being semi-impervious; the latter impervious. All are well graded from gravel sizes through silt sizes with some clay. Occasional boulders are encountered. The overlying sediments are quite variable in composition including sands, silty sands and gravelly sands with occasional boulders generally concentrated at the surface. Plate II-9 is a plan of borrow exploration and Plate II-10 is a record of selected representative explorations.

c. Construction Materials.- (1) Materials Required.- Materials required for the construction of the dam are summarized in the following tabulation:

Item	Quantity - C.Y.
Compacted Impervious	148,000
Compacted Random	195,000
Compacted Pervious	200,000
Semi-Compacted Random Fill	174,500
Structure Backfill	13,000
Special and Processed Aggregate	175,000
Rock for Slope Protection	90,500

(2) Materials Available in Required Excavation.- Required excavation for the cutoff trench, spillway, approach channel, and discharge channel involves the removal of approximately 658,000 cubic yards of material. Of this amount, 172,000 cubic yards (approximately 25%) is stripping and waste material. The remaining 486,000 cubic yards of material is considered suitable for use in the embankment for the following purposes:

Item	Quantity - C.Y.
Impervious Fill	23,000
Random Fill	294,000
Pervious Fill	144,000
Rock for Slope Protection	25,000
Selected Stripping for Upstream Cofferdam	53,000

Rock for slope protection consisting of boulders obtained from structure excavation is estimated at five per cent of total usable excavation.

(3) Materials Available for Borrow.- (a) Impervious Borrow.- The most suitable locally available sources of impervious borrow are the massive deposits of glacial till. A total of five different areas were investigated as possible sources of impervious borrow as shown on Plate II-9. The most economical deposit and the one selected as a source of impervious borrow for the embankment is Area A located approximately 2,000 feet northeast of the spillway location and is a continuation of the deposit that forms the east abutment of the dam. The borrow area consists of a side hill with rather irregular topography cut by a small creek with intermittent flow. The material in the deposit is a well graded gravel, sand and silt with some clay sizes and occasional boulders. The thickness of the material suitable for impervious borrow ranges from 10 feet to greater than 25 feet. Ground water observations taken during drilling operations in the till indicate the water table during the fall and winter is at a depth of about 5 feet. The installation of observation wells at several locations in the borrow area will be made as soon as possible. Overlying the till is a layer of material approximately 15 feet in thickness, which is classified as random borrow and is used in the embankment as described in the following subparagraph.

(b) Random Borrow.- The random borrow consists of approximately the top 15 feet of semi-impervious to pervious soil which overlies the impervious borrow area described in the preceding paragraph. Excavation of this material is required to obtain impervious borrow. Random material is used in the structure in the semi-compacted fill sections. The material consists of variable sands and sandy till and contains occasional cobbles and boulders.

(c) Pervious Borrow.- There are two principal types of deposits which are suitable for pervious fill in the embankment: (1) The chain of eskers which lie principally on the east side of the river and extend for about a mile starting in the vicinity of the Powder Mill Dam and (2) the extensive terrace deposits which form the western edge of the valley. The eskers are approximately

25 feet in height and consist generally of well graded clean sand, gravel and cobbles with pockets and lenses of clean sand and sandy gravel. The terrace deposits consist of clean medium to coarse sand with gravelly phases. Both the eskers and the terrace deposits have been worked as borrow pits for local roads and other construction. Both deposits are close to the dam site and explorations for pervious borrow have been confined to these two. Ground water in the pervious deposits is below the proposed depth of excavation. Stripping of one to two feet of sandy topsoil is required to expose usable material.

(d) Processed Aggregates.- A survey to locate all possible commercial sources of processed aggregate is in progress. Information obtained to date indicates that processing plants are now operating at the following locations:

Location	Approximate Haul Distance
Manchester, N. H.	30 mi.
Keene, N. H.	30 mi.
Fitchburg, Mass.	45 mi.

Since all of these plants are at a considerable haul distance, it is considered economical to establish a plant at the site to process aggregates from material excavated from the pervious borrow area. Because of the substantial proportion of oversize available, crushing of oversize is proposed. The suitability of aggregates processed from this source has not been determined to date; however, experience with similar deposits indicates that, with controlled washing and sizing, satisfactory aggregates may be obtained.

(e) Rock for Slope Protection.- Two principal sources of rock for slope protection are being considered: (1) The separation and use of cobbles and boulders between one-half cubic foot and one cubic yard in size available from structure excavation, impervious borrow excavation, random borrow excavation augmented by collection of surface boulders within project area and stone fences in vicinity. (2) The establishment of a rock quarry area at Bell Ledges located on Plate II-9. Estimates indicate that approximately two-thirds of the total rock required may be obtained by separation and collection of boulders. These boulders are composed almost without exception of granite and are sound and unweathered except on exposed sides where shallow surface weathering may be observed. The large exposures of bedrock at Bell Ledges consist of porphyritic granite. The formation at this location is suitable for quarrying of sound rock without appreciable overburden removal. Approximately one mile of access

road is required to the site. The haul distance is three miles, all downhill.

In addition to these two principal sources, consideration is being given to the designation of a quarry area at the spillway site of the West Peterboro Reservoir. The site for the spillway which is now under consideration by this office is located in a saddle above Half Moon Pond, approximately 6 miles from the Bennington Dam site. Quarrying from this site involves considerable overburden removal. Boulders which are frequent on the ground surface at this site may be collected and added to the rock excavated. Because of the greater haul distance and the necessary overburden removal, quarrying from this site will prove economical only if the construction of the West Peterboro project is authorized.

(4) Materials Summary.- A summary of construction materials required with indicated source, and materials available from required excavations and selected borrow areas with indicated disposition is shown in the tabulation below. Indicated quantities include allowance for shrinkage and settlement.

CONSTRUCTION MATERIALS

MATERIALS REQUIRED FOR EMBANKMENT

Item	Quantity - C.Y.		Source
	Embankment Measure	Excavation Measure	Excavation Measure C.Y.
Compacted Impervious Fill	148,000	170,000	18,000 Structure Excavation 152,000 Impervious Borrow
Compacted Random Fill	195,000	225,000	225,000 Structure Excavation
Compacted Pervious Fill	200,000	230,000	144,000 Structure Excavation 86,000 Pervious Borrow
Semi-compacted Fill	174,500	200,000	59,000 Structure Excavation 53,000 Selected Stripping 88,000 Random Borrow
Structure Backfill	13,000	15,000	5,000 Imp. Structure Excavation 10,000 Random Structure Excavation
Special and Processed Gravel	175,000	210,000	208,000 Pervious Borrow
Rock for Slope Protection	90,500	70,000	14,000 Quarry 56,000 Oversize from Excavation

MATERIALS AVAILABLE FROM EXCAVATION AND BORROW

Item	Quantity - C.Y.	Disposition - C.Y.
Stripping Foundation	168,000	53,000 Upstream Cofferdam 115,000 To Spoil Areas
Structure Excavation		
Impervious	25,000	2,000 Oversize to Riprap 5,000 Structure Backfill 18,000 Compacted Impervious Fill
Random	309,000	15,000 Oversize to Riprap 225,000 Compacted Random Fill 59,000 Semi-compacted Fill 10,000 Structure Backfill
Pervious	152,000	8,000 Oversize to Riprap 144,000 Compacted Pervious Fill
Total Usable Excavation	486,000	
Impervious Borrow	165,000	5,000 Waste 8,000 Oversize to Riprap 152,000 Compacted Impervious Fill
Random Borrow	110,000	14,000 Stripping and Waste 8,000 Oversize to Riprap 88,000 Semi-compacted Fill
Pervious Borrow	350,000	39,000 Stripping and Waste 15,000 Oversize to Riprap 210,000 Special & Processed Gravel 86,000 Compacted Pervious Fill
Quarry Stone	20,000	6,000 Waste 14,000 Riprap

d. Soil Data.- (1) Scope and Extent of Laboratory Investigation.- All samples of material encountered in field exploration have been submitted to the laboratory for final classification and testing. Selected representative samples have been tested to determine compaction characteristics, shear strength, permeability and consolidation characteristics. Investigation of these soil properties is continuing but sufficient data have already been obtained to determine the general range of quantitative results.

(2) Laboratory Procedures.- (a) Mechanical Analysis.- Mechanical analysis of selected representative samples has been made using a standard sieve analysis with a minimum sieve of 100 meshes per inch, and hydrometer analysis of all sizes passing that sieve.

(b) Specific Gravity.- The specific gravity of principal materials has been obtained by the water displacement method (A.A.S.H.O. T100-38).

(c) Density.- Density of soils having cohesion has been determined from undisturbed chunk samples. Density of cohesionless soils has been determined in the field by the sand displacement method.

(d) Water Content.- The natural water content of principal materials was determined from samples obtained in the field and transported to the laboratory in parafined jars. Results are reported in terms of percentage of oven dry weight.

(e) Compaction Characteristics.- Compaction characteristics of the cohesive soils have been determined by Modified Proctor tests using a ten-pound hammer, 18-inch drop and 5 layers of soil. Compaction characteristics for cohesionless soils include determination of minimum dry density by placing soil in a container without vibration or impact, and maximum dry density obtained by impact compaction with complete saturation.

(f) Shear Strength.- Shear strength of principal materials was determined in accordance with procedures outlined by A. Casagrande and R. E. Fadum, in "Notes on Soil Testing for Engineering Purposes," a publication from Harvard University Graduate School of Engineering.

(g) Permeability.- Permeability of materials was determined using de-aired water in a falling head type apparatus with a plastic, transparent permeameter following the general recommendations of G. E. Bertram in "An Experimental Investigation of Protective Filters," a publication of Harvard University Graduate School of Engineering.

(h) Consolidation.- Laboratory consolidation characteristics are determined by using fixed ring consolidation test apparatus for a 4-1/4-inch diameter sample of 1-1/4-inch initial thickness.

(3) Test Results.- (a) Classification of Materials Encountered.- Classification of materials encountered in field explorations are shown with the graphic logs of explorations on Plates II-4 through II-8 and Plate II-10. This classification includes color, compactness, consistency, plasticity, and basic soil type of each stratum encountered.

(b) Soil Data Summary.- A summary of data, compiled from laboratory tests performed on samples of principal soil strata, appears on Plate II-11. On this plate Figures 1 and 2 show range of gradation of the foundation and borrow materials, respectively. Figure 3 is a moisture-density curve for the silty till from the spillway foundation. The impervious borrow for the core of the dam is from another portion of this same geological formation. Curves showing the shear strength for silty till and for the pervious materials are shown on Figures 4 and 5, respectively. Figure 6 is a summary of properties of materials encountered in required excavation and in the borrow areas. Laboratory investigation to determine soil properties is incomplete. Conservative estimated values shown in Figure 6 are based on results of tests on similar materials for other sites, particularly at Franklin Falls and Blackwater Dams.

(4) Investigations in Progress.- (a) Compaction, Shear and Permeability Tests.- Preliminary tests to determine compaction characteristics, shear strength and permeability values of all principal materials have been performed on typical soils. Additional tests are in progress to determine the range of data for variation in the soils.

(b) Filter Design.- Preliminary filter design was based entirely on criteria outlined in Chapter XXI of the Military Engineering Manual, published by the Chief of Engineers. Ultimate filter design will be based on laboratory tests of construction materials.

(c) Suitability of Material for Concrete Aggregates.- There are no commercial aggregate plants of adequate capacity in the general vicinity of the project, but concrete aggregates are available by processing material from the pervious borrow area. Experience of this office and other Federal and State agencies indicate that these clean glacial gravel deposits are excellent sources of concrete aggregate. Samples of this pervious material have been obtained for complete analysis at the North Atlantic Division Central Concrete Laboratory.

c. Embankment Design.- (1) Design Criteria.- The design of the dam embankment and dam embankment foundation involved a study of the foundation conditions and characteristics, a study of the characteristics of the available embankment materials, the choice of a section which utilizes economically the available embankment materials and is safe under any condition. This section includes a description of the results of those investigations which are pertinent to the design, a discussion of the choice and economy of the embankment section, and the analysis demonstrating that the section is satisfactory for the following criteria;

(a) The slopes of the embankment must be such that no shear slide can occur in the embankment or foundation materials.

(b) The void ratio of all materials in which a flow slide might occur must be less than the critical void ratio.

(c) Seepage must be controlled so that no detrimental uplift pressures or transportation of material can occur.

(d) Provision should be made to compensate for the settlement of the embankment after construction to insure the design free board height.

(2) Preliminary Embankment Design.- Several preliminary project designs have been considered for various alignments within the general project area. The present alignment was selected as the most feasible and the embankment design presented for review by the Board of Consultants in December 1944 was essentially the same as reported herein. The one change recommended by the Board of Consultants and incorporated in the design is a substantial reduction in width of the upstream impervious blanket on the west abutment. If additional width is required after construction to control seepage in this area, the blanket may be increased readily.

(3) Definite Project Design Features.- (a) Foundation Conditions.- Foundation conditions have been described for five different areas of the dam site (Paragraph b.).

(b) Compacted Impervious Section.- The compacted impervious section of the embankment consists of a central core which ties into the till body on the east abutment and across a large portion of the valley. On the west side of the valley where no till occurs in the foundation and where till is buried too deeply for economical contact the portion of the central

impervious core below the stripping line is reduced and supplemented by an impervious blanket under the upstream portion of the embankment. This blanket extends slightly beyond the upstream toe of the dam.

(c) Compacted Random Sections.- The compacted random sections of the embankment have been included in the design to provide a transition between the impervious core and the pervious section of the dam. Dimensions of these random sections have been chosen to utilize the estimated quantity of suitable material from structure excavation at the dam site and required random excavation in the impervious borrow area.

(d) Compacted Pervious Sections.- The compacted pervious sections of the dam have been designed to provide sufficient stability of the upstream section during rapid drawdown of water elevation and to hold the maximum line of seepage in the downstream sections well below ground surface for conditions of sustained high upstream pool elevation.

(e) Special Drainage Features.- Drainage of water seeping through and beneath the earth embankment is collected by a filter blanket beneath the downstream portion of the embankment with perforated pipe drain and a drainage well system. The drainage wells are located in the area where subsurface drainage of the pervious substratum is restricted by closure of the upper and lower till layers. Details of the filter blanket with perforated pipe drain are shown on Plate IV-2 and details of the drainage well design are included on Plate II-12.

(f) Slope Protection.- The upstream slope of the earth embankment is protected from wave action and surface erosion by a dumped rock fill laid upon a layer of screened gravel as shown by details on Plate II-12. The downstream slope up to 7 feet above maximum tail water elevation is protected similarly. Above the rock fill on the downstream slope, the slope is protected against surface erosion by a layer of sand gravel and cobbles with the coarse sizes pulled to the surface by raking.

(4) Design Studies.- (a) Design of Filters.- In all sections of the embankment and its foundation through which water passes, a study has been made to insure that the gradation of adjacent soils is such that the finer sizes of one soil will not be transported by the seeping water into the voids of an adjoining soil. In areas where seepage water is collected, the gradations of adjacent soils fulfill the above criteria and in addition

the soils become progressively several times more pervious in the direction of discharge. The critoria used for these analyses is that contained in Chapter XXI of the Military Engineering Manual, OCE. The determination of the general range of material for the filter blanket in the downstream section of the earth embankment is illustrated by Plate II-14.

(b) Study of Seepage.- The seepage through and beneath the earth embankment has been studied by flow nets, Plate II-12. Based upon these flow nets, the total seepage through and beneath the embankment, including spillway section, is estimated at one-half c.f.s. for maximum head, ultimate section. Of this total, a negligible portion will be discharged beneath the spillway into the collector system and 0.15 c.f.s. through the drainage well system. The greater portion of the remaining seepage will occur beneath the western portion of the embankment in the section containing the upstream impervious blanket.

(c) Embankment Stability.- (1) Method of Analysis.- Using the most dangerous circle method, the stability ratio for shear failure of the dam embankment and its foundation was determined by investigating the forces tending to cause movement and those producing potential resistance to movement on several circular sliding surfaces of weakness which were selected by systematic trial. This method investigates only the possibility of a shear failure. The analysis of flow slide failure is described in a following subparagraph. In the analysis the driving forces include the rotating effect of the weight of the soil mass and water above the surface of failure and also the forces generated by water pressure. The forces producing potential resistance consist of the shear strength generated along the sliding surface. The ratio of the potential resisting force and the driving force is termed the stability ratio. A sufficient number of potential surfaces were analyzed to determine the position of the surface having the least stability ratio, termed the "minimum" stability ratio. A minimum stability ratio of unity indicates equality of driving and potential resisting forces and implies that the embankment is on the verge of failure, while a minimum stability ratio of greater than unity indicates that the structure possesses reserve strength.

(2) Section Analyzed.- Analysis was made of the assumed section for ultimate development which includes the section designed for the initial development. It is known by experience that the initial section will have a minimum stability ratio, by the method used, equal to or greater than the minimum stability ratio of the assumed maximum section for the ultimate development. In the analysis the embankment and its foundation were considered to be integral and sliding surfaces were allowed to pass through foundation and embankment sections without discrimination. The soil characteristics used in the analyses of the various sections are contained on Plate II-11.

(3) Upstream Slope.- The results of the investigations of several potential failure planes in the upstream section of the dam embankment are shown on Plate II-13. The minimum stability ratio considering the embankment and its foundation as a whole, and for the conditions existing immediately after a sudden draw down of upstream pool is 1.80. Analyses were made using the method of slices. The usual approximation was made that the soil forces on either side of each slice balance each other. It is assumed that the rock slope protection, the gravel bedding, the compacted pervious and compacted random sections will drain as rapidly as the pool is drawn down and that the compacted impervious section will not drain and is saturated. For the failure surface with minimum stability ratio, and assuming that the compacted random section does not drain, the stability ratio decreases to 1.7.

(4) Downstream Slope.- The results of the analysis of the downstream section of the dam embankment for the ultimate development are shown on Plate II-13. Minimum stability ratio is 1.58 for conditions of steady seepage.

(5) Foundation and Abutments.- Stability of the foundation against shear failure was determined in conjunction with the analyses of the upstream and downstream slopes. The abutments and the transitions between earth embankment and spillway are considered of greater stability than the principal embankment section.

(d) Flow Failure Analyses.- A flow failure is defined as the liquefaction by shock of a mass of loose, saturated cohesionless material. Based upon the experience gained by the detailed study of flow failure made in connection with the design of the Franklin Falls Dam, a flow failure of either the upstream compacted pervious or random sections of the embankment or the embankment foundation is considered highly improbable. A detailed analysis of the possibility of a flow failure will be made in connection with the final design.

(e) Surface Slides.- Surface slides may occur during the period of frost melting in soils affected by frost action. Such surface slides will not occur in the embankment since all materials in the range of frost penetration (at this site about four feet) are cohesionless and not susceptible to frost action.

(f) Settlement Analysis.- Based upon the experience gained as a result of settlement observations on the completed Franklin Falls Dam, the settlement of the compacted embankment at its maximum section is expected to be approximately two to six inches due to foundation consolidation, which will occur simultaneously with construction, and one to four inches due to consolidation of the compacted impervious section under its own weight, which settlement will occur gradually over an extended number of years. After further analyses have been made definite values will be determined for the latter settlement and the embankment constructed to the design height plus the settlement allowance.

f. Spillway and Outlet Structure Foundation.- (1) Selection of Location.- The location of the combined spillway and outlet structure at the site shown was selected after considerable subsurface exploration work had been carried on in the general vicinity. A location with bedrock at an elevation sufficiently high to provide economical construction could not be located; hence, the structures are founded on overburden. The most suitable overburden at the site for foundation of the structures is the silty till, because it is both the most impervious and the most compact. The location selected was chosen because:

(a) The unconsolidated sediments over the silty till are relatively thin compared to other locations considered.

(b) The silty till is continuous to bedrock while at some other sites considered it was discontinuous and underlain by more pervious and more compressible soils.

(2) Conditions Encountered.- At the selected spillway and outlet structure site, the subsurface conditions encountered are described in Paragraph 2 and shown on Geological Profiles 1, 2 and 3 of Plate II-3.

(3) Foundation Design.- (a) Design Details.- The spillway and outlet structure are founded directly upon the compact silty till excavated to approximately elevation 649. To form a working surface upon the excavated till and to protect it from becoming loosened, approximately one foot of

concrete will be placed as soon after excavation as practical. The surface of this concrete will be left rough. The downstream toe and the stilling basin slabs will be founded upon a drainage filter which will consist of a layer of sand laid upon the till then a layer of screened gravel. A system of open joint pipe will be laid in the gravel layer (Plate IV-3) A layer of porous concrete approximately one foot thick will be placed upon the filter for protection and to provide a working surface and to facilitate drainage of seepage entering the filter layer. The concrete slabs will be placed directly upon the porous concrete layer.

(b) Stability against Sliding.- The analysis for the stability of the spillway and outlet structure against sliding along its base is contained in Appendix IV. This analysis assumes that the friction between the concrete and compact silty till is equal to the shearing resistance of the silty till. The shear resistance of the silty till as determined by the tests upon remolded samples is summarized on Figure 4, Plate II-11. For the stability analysis, the shear strength has been conservatively assumed as represented by an angle of internal friction of 35° with zero shear strength at zero normal load. The analysis includes full consideration of uplift forces acting upon the structure due to conditions of steady seepage (Plate II-13). No allowance is made for the substantial restraint against sliding offered by the earth embankment abutments.

(c) Seepage Beneath Structure.- Seepage beneath the structure is collected by the drainage filter beneath the downstream toe and stilling basin. The seepage water is collected by the open joint drains which discharge into the stilling basin through walls in the side walls. The total quantity of seepage which will be discharged into this system is estimated at 0.0001 c.f.s. for ultimate maximum upstream pool height based upon the flow nets on Plate II-13. The maximum hydraulic gradient for flow into the drainage filter is approximately 0.85, which is considered sufficiently less than the critical maximum value of 1.0 to be amply safe assuming no unusual local discontinuities.

(d) Settlement of Structure.- Based upon experience with concrete structures founded upon very compact till, it is considered that the total settlement, due to consolidation of the till under the weight of the structure, will be in the order of one inch. This settlement is not considered excessive and the structure should not be impaired by such a settlement.

(4) Problems During Construction of Spillway.-

(a) Seepage Into Excavation.- The hydrostatic head in the zone of pervious weathered bedrock beneath the spillway structure causes water to rise in observation wells to approximately elevation 676. The deepest excavation will be to elevation 646, which will result in upward seepage from bedrock into the excavation under a head of 30 feet assuming the hydrostatic head in the bedrock remains unchanged. The thickness of compact silty till between bedrock and the bottom of excavation will be between 25 and 45 feet. The hydraulic gradient for upward seepage will be from 0.7 to 1.2. In cohesionless soils, a vertical component hydraulic gradient of approximately one or greater is considered unsafe and may result in boils and quicksand action. Since the silty till possesses a slight cohesion and exists in a very compact state, it is considered possible that a hydraulic gradient as great as 1.2 may not result in detrimental action. Since it is exceedingly important from the standpoint of the settlement of the structure that no loosening of the till occur by upward seepage, funds have been requested for performing a pumping test to ascertain the difficulties of lowering the hydrostatic head in the bedrock. The necessity for reduction of hydrostatic head in the bedrock is considered of basic importance and pending the above test funds are included in the cost estimate to cover pumping of water from the bedrock at a minimum of four, large diameter deep wells during construction operations.

(b) Frost Action.- The foundation will be completed and sufficient concrete placed over the foundation to prevent frost penetration into the glacial till in the winter between the two construction seasons.

(c) Stability of Excavation Slopes.- A number of analyses have been performed to determine the steepest slope that may be made for the excavation for the spillway and outlet structure. Based upon the results of shear strength tests, Paragraph (d), the slopes shown on Plate II-13 have been tentatively selected. The stability ratio for these slopes is approximately 1.2 assuming a condition of sudden drawdown of water in the excavation. For this condition, a stability ratio of 1.2 is considered adequate. Based upon experience with other similar till deposits, it is considered that a steeper slope may be satisfactory. The slope will be protected during the fall, winter and spring to prevent damage through frost action and surface erosion.

g. Approach and Discharge Channels.- (1) Design Details.- The approach and discharge channels have been designed of size to pass the required quantities of water (Appendix I). The side slopes, which vary between 1 on 2 and 1 on 2½ have been designed to be stable under conditions of rapid drawdown of water surface in the channel. Where appreciable water velocities occur a protective riprap placed upon a gravel filter has been provided. The thickness and size of riprap has been increased in the zone of unusually high and erratic water velocities immediately downstream from the stilling basin. The details of slopes and slope protection are illustrated on Plate IV-3.

(2) Disposition of Excavated Material.- The materials which will be excavated from the spillway discharge and approach channels will be used if suitable in the compacted fill in the embankment or in grading operations at the dam site as described in Paragraph C.

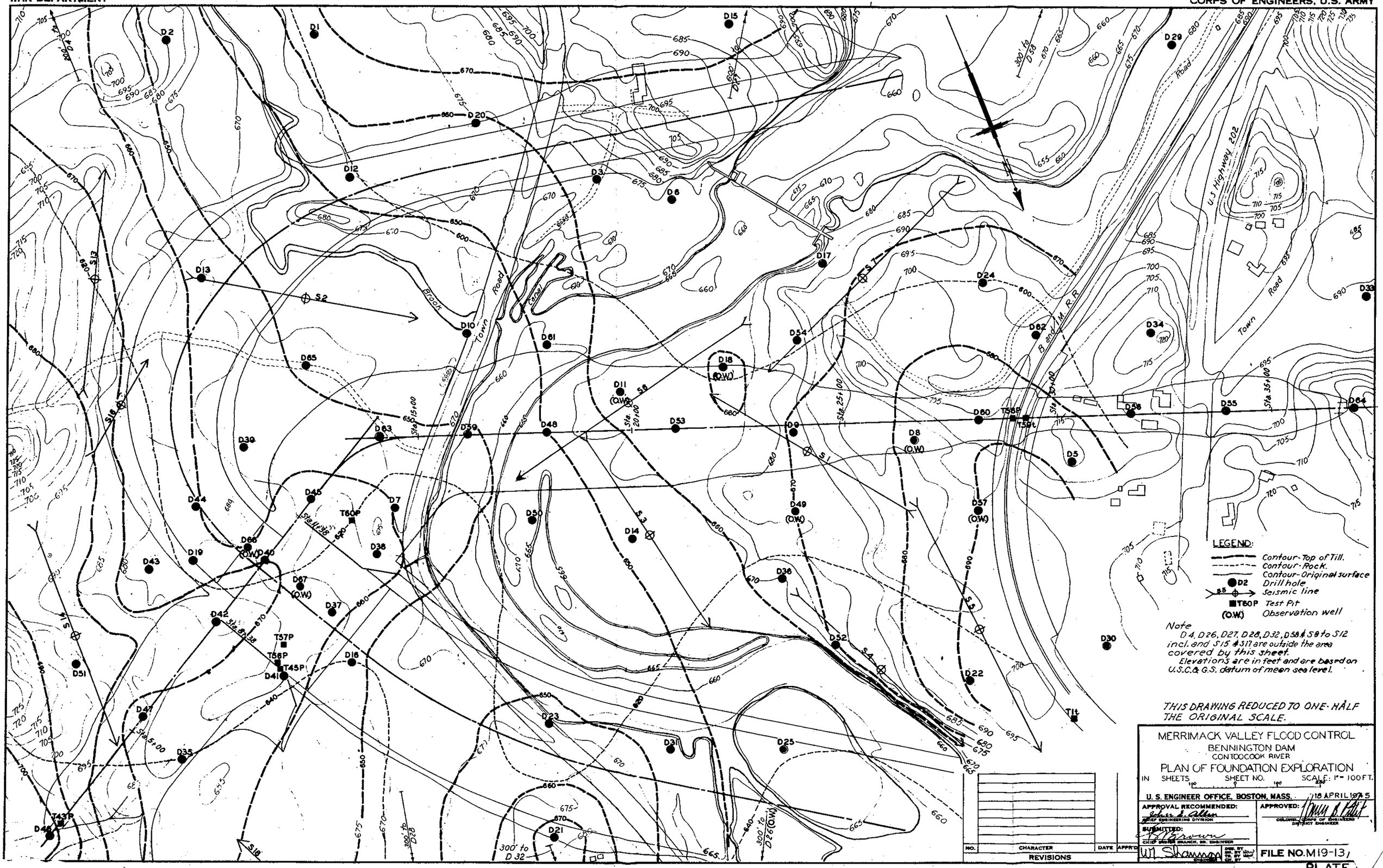
h. Upstream Cofferdam Design.-(1) Location and Description.- The upstream cofferdam extends from the embankment at the west side of the spillway to high ground of the west abutment. The cofferdam consists of an earth embankment of 20 feet top width, approximately 25 feet high in the river section and approximately 15 feet high in the land sections (Plate II-15). The upstream side slopes of the cofferdam are 1 on 2.5 and downstream side slopes are 1 on 2. The embankment is composed of selected stripping and random materials from structure excavation. A downstream toe drain of coarse bank run gravel is provided to prevent erosion by water seeping through or under the structure.

(2) Stability Analysis.- A stability analysis by the dangerous circle method has been made for the upstream slope of the river section for the condition of sudden drawdown of water from the design maximum elevation of 685 to elevation 667 (Plate II-15). For this case a stability ratio of 1.14 was obtained which is considered sufficient for this temporary structure.

(3) Seepage Analysis.- Analysis of quantity of seepage both through and under the cofferdam in the river section and on the land section has been determined by construction of flow nets for these two cases (Plate II-15). The total quantity of seepage that may be expected if the maximum upstream water surface is maintained for sufficient time to develop full seepage through the embankment will be approximately 0.02 c.f.s. From the proportions of the flow net the required resistance to uplift or flotation at the downstream toe of the cofferdam is assured with a maximum discharge gradient of approximately 0.67 in comparison with a maximum allowable gradient of 1.0.

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



LEGEND:

- Contour-Top of Till.
- Contour-Rock.
- Contour-Original surface
- D2 Drill hole.
- S5 Seismic line
- T60P Test Pit
- (O.W.) Observation well

Note

D4, D26, D27, D28, D32, D58 & S8 to S12 incl. and S15 & S17 are outside the area covered by this sheet.

Elevations are in feet and are based on U.S.C.& G.S. datum of mean sea level.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

MERRIMACK VALLEY FLOOD CONTROL

BENNINGTON DAM

CONTOOCOOK RIVER

PLAN OF FOUNDATION EXPLORATION

IN SHEETS SHEET NO. SCALE: 1" = 100 FT.

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: John D. Pitts, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN BRANCH, SR. ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

REVISIONS

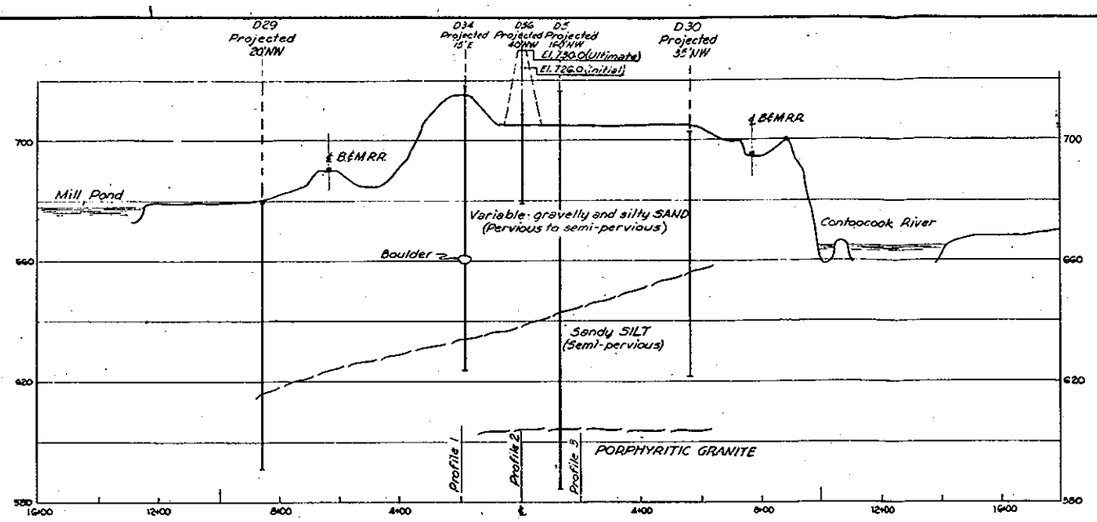
NO.	CHARACTER	DATE	APPR'D

FILE NO. M19-13)

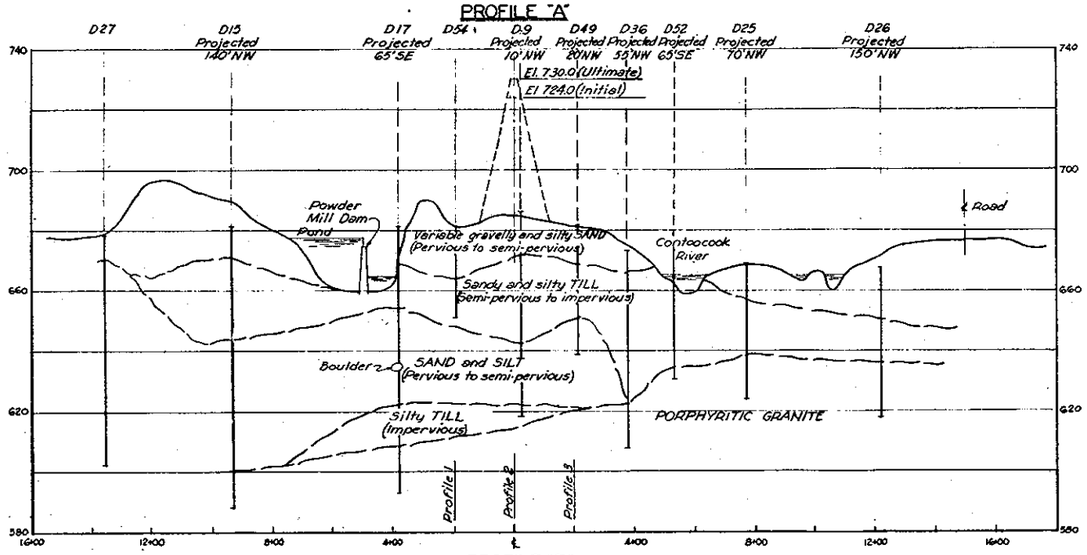
PLATE 1

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



PROFILE "A"



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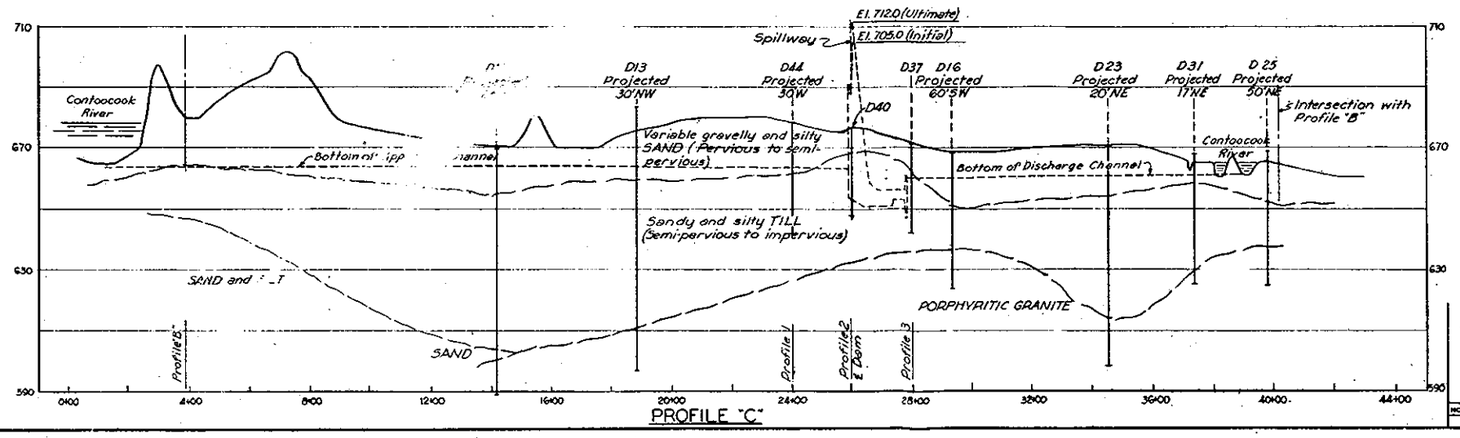
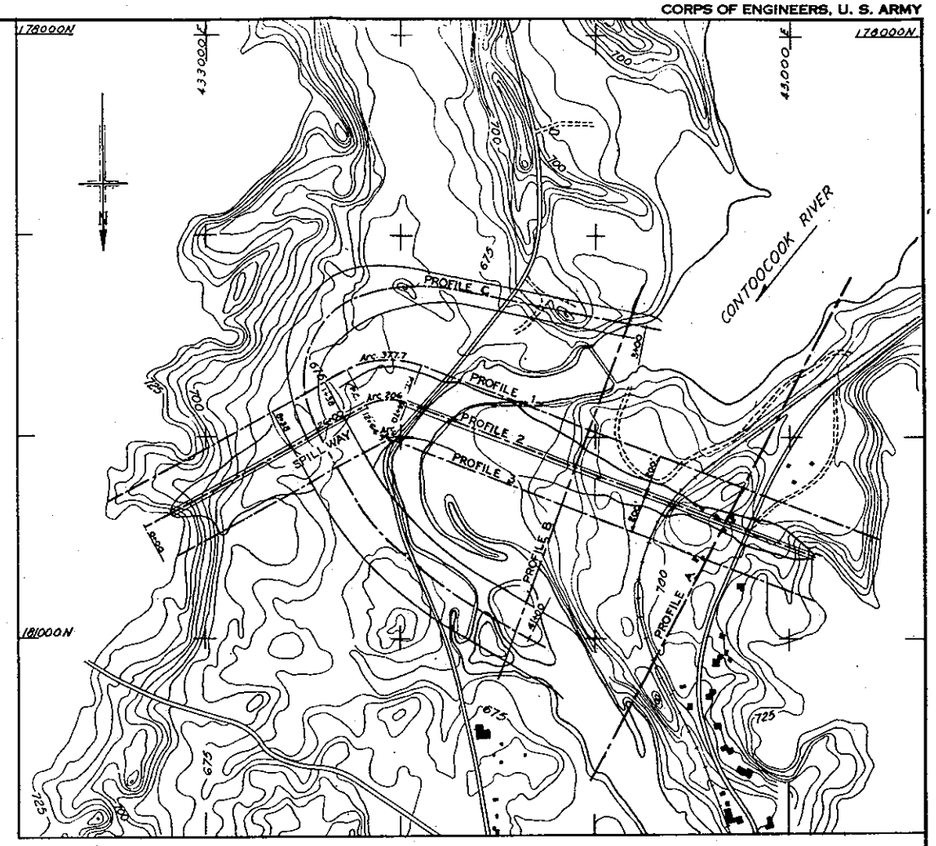


PROFILE "C"



KEY PLAN

Scale: 1"=300'

300 0 300 600 900 FT.

SCALE

NOTE:
Elevations are based on U.S.C. & G.S. Datum of Mean Sea Level.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

GEOLOGICAL PROFILES - NO. 1.

IN SHEETS SHEET NO. SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED

SUBMITTED: W.L. Shannon

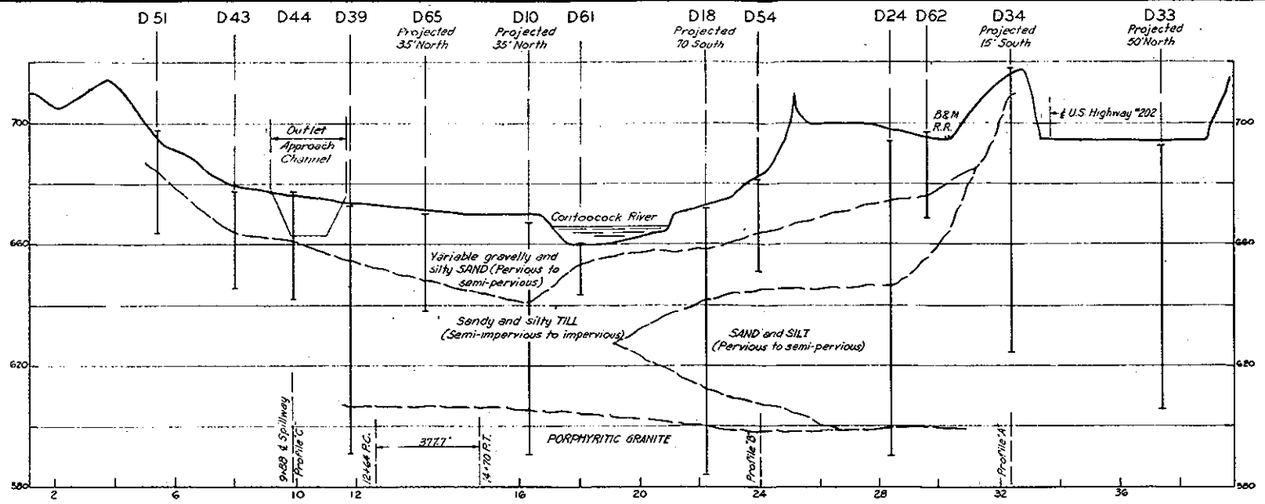
APPROVED:

DES. BY
DR. BY
TR. BY
CH. BY

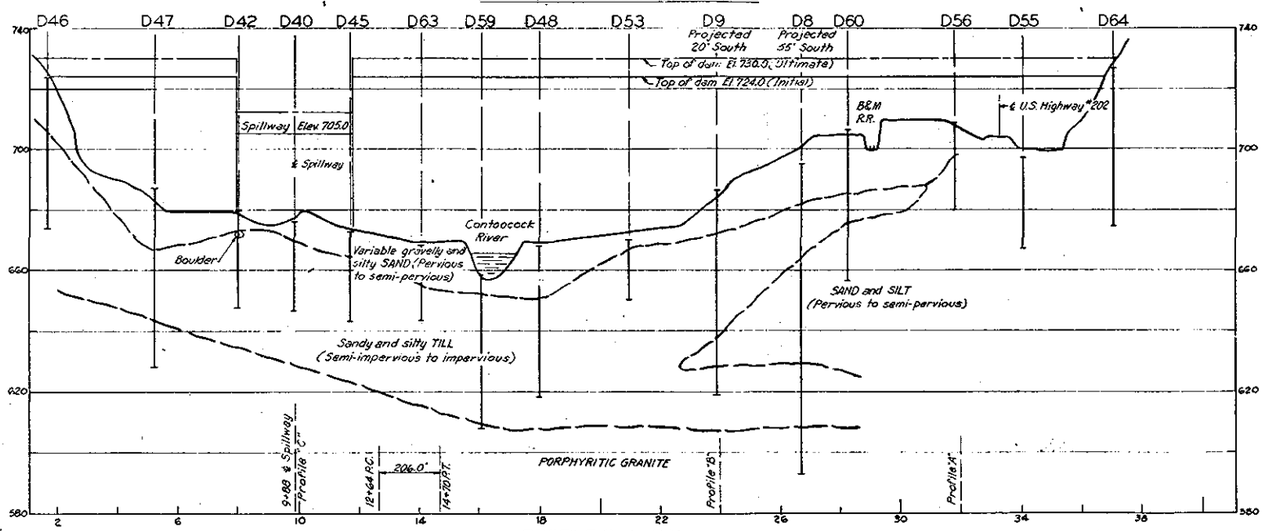
FILE NO. M19-13/24

NO. CHARACTER DATE BY
REVISIONS

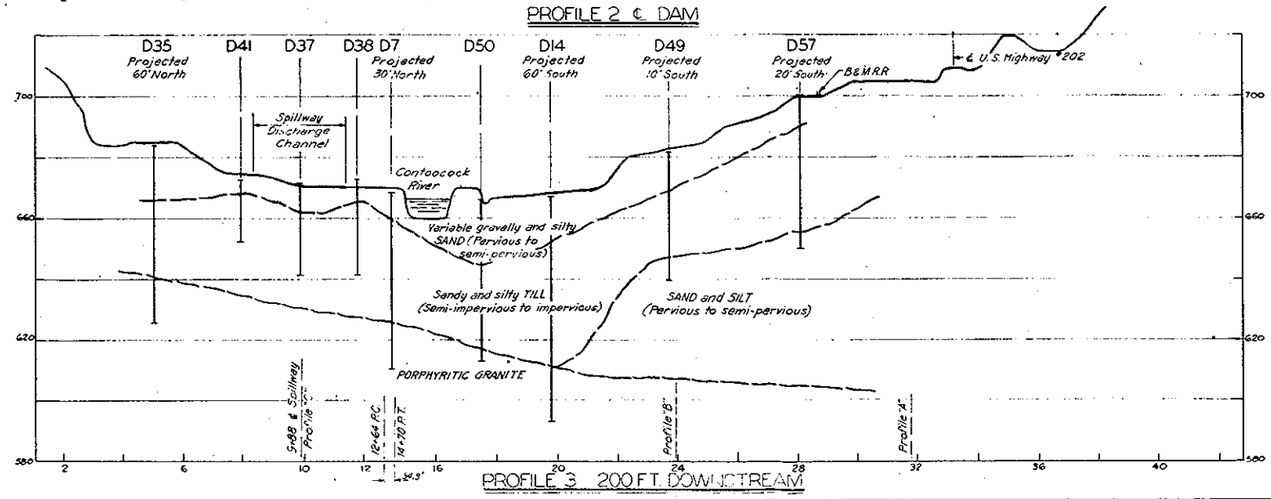
PLATE II-2



PROFILE 1 200 FT. UPSTREAM



PROFILE 2 ℄ DAM



PROFILE 3 200 FT. DOWNSTREAM

NOTE:-
For location of profiles see key plan on Plate II-2.
Elevations are based on U.S.C.& G.S. Datum of Mean Sea Level.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

GEOLOGICAL PROFILES-NO. 2

IN SHEETS SHEET NO. SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: [signature], CHIEF, DESIGN SECTION, DIV. ENGINEER

[signature], ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M 19-13/25

NO.	CHARACTER	DATE	BY
	REVISIONS		

D1

673.3 0.0
671.8 1.5 Sandy TOPSOIL
668.4 4.9 Brown, loose, cohesionless, fine silty SAND

Gray, compact, cohesive poorly graded, gravelly and silty SAND. (Silty TILL)

633.3 40.0

D2

682.6 0.0
681.1 1.5 Sandy TOPSOIL
677.6 5.0 Brown, loose, cohesionless, coarse to fine silty SAND

Brown, loose, cohesionless, uniform coarse SILT with coarse to fine silty sand layers.

648.0 34.6
642.6 40.0 Gray, very compact, cohesive, poorly graded, gravelly and silty SAND (silty TILL)

D3

675.0 0.0
674.5 0.5 Highly organic TOPSOIL
672.0 3.0 Gray, semi-compact cohesionless, coarse to fine, gravelly SAND
Groundwater, 24 Aug. 1942)
663.5 11.5 Light brown, semi-compact, cohesionless uniform sandy SILT containing cobbles and boulders.

Brown, compact, cohesive coarse to fine, gravelly and silty SAND. (silty TILL)

632.9 42.1

Gray, compact, cohesionless, coarse to fine, gravelly and silty SAND.

613.9 61.1

Light brown, very compact, slightly cohesive, gravelly and silty SAND. (Sandy TILL)

601.5 73.5

Light brown, compact slightly cohesive, coarse to fine silty SAND. Boulder drilled from El. 597.0 to 593.0

584.2 90.8

D4

710.0 0.0

Brown, loose, cohesionless. coarse to fine SAND.

678.5 31.5

Brown, loose, cohesionless, gravelly SAND.

666.0 44.0

Gray, compact, slightly cohesive SILT. (Occasional cobble and boulder)

645.1 64.9

PORPHYRITIC GRANITE fractured, and weathered along joints.

630.7 79.3 Rock core recovery 44%

D5

715.0 0.0
714.0 1.0 TOPSOIL

Light brown, loose, cohesionless, fine SAND.

698.2 16.8

Brown, loose, cohesionless, coarse to fine SAND.

688.0 27.0

Brown, compact, cohesionless poorly graded, gravelly, silty SAND, with cobbles. (sandy TILL)

675.0 40.0

Brown, compact, cohesionless SILT.

601.7 113.3

Decomposed GRANITE.

583.7 131.3 Rock core recovery 10%

D6

673.0 0.0
672.0 1.0 Sandy TOPSOIL

Brown, compact, cohesionless gravelly SAND.

663.0 10.0

Brown, compact, cohesionless gravelly, silty SAND (sandy TILL)

656.0 17.0

Gray, very compact, slightly cohesive, gravelly, silty SAND (sandy TILL)

630.2 42.8

Brown-gray, very compact, cohesionless, silty SAND Artesian flow encountered at El. 617.8.

606.6 66.4

D7

668.7 0.0
667.2 1.5 TOPSOIL

Brown, loose, cohesionless, uniform medium to fine SAND.

659.7 9.0

Gray, very compact, cohesive coarse to fine, gravelly and silty SAND. (silty TILL) Boulder drilled from El. 655.1 to 654.1

625.7 43.0

PORPHYRITIC GRANITE with shallow weathered surface

610.7 58.0 Rock core recovery 86%

D8

695.1 0.0
694.8 0.3 TOPSOIL

Brown, loose, cohesionless, uniform, medium to fine.

687.1 8.0 SAND

Gray-brown, loose cohesionless, gravelly SAND.

681.9 13.2

Brown, compact, slightly cohesive coarse to fine SAND (sandy TILL)

671.1 24.0

Gray, compact, cohesionless, medium to fine silty SAND. Boulder drilled from El. 665.1 to El. 663.1

655.8 39.3

Brown, compact, cohesionless, coarse to fine silty SAND.

633.3 61.8

Gray, compact, slightly cohesive sandy SILT.

628.1 67.0

Gray, very compact, slightly cohesive coarse to fine SAND. (silty TILL). Boulder drilled from El. 627.8 to El. 626.8.

608.6 86.5

Decomposed ROCK

592.6 102.5 No rock core recovered Observation well installed.

D9

686.4 0.0
685.9 0.5 TOPSOIL
682.4 4.0 Brown, loose, cohesionless, uniform, fine gravelly SAND.

Brown, compact, cohesionless, coarse to fine gravelly SAND.

671.9 14.5

Gray, very compact, slightly cohesive, coarse to fine, gravelly, silty SAND. (silty TILL)

637.6 48.8

Brown, compact, cohesionless, medium to fine SAND

628.9 57.5

Gray, compact, cohesionless, fine silty SAND

622.4 64.0
619.1 67.3 Gray, very compact, slightly cohesive coarse to fine, gravelly, silty SAND. (silty TILL)

D10

667.2 0.0
665.4 1.8 Organic TOPSOIL
Groundwater, 16, June 1944.

657.4 9.8

Gray, loose, cohesionless, fine silty SAND.

640.2 27.0

Gray, compact, slightly cohesive, poorly graded, gravelly, silty SAND with numerous cobbles. (silty TILL)

605.2 62.0

PORPHYRITIC GRANITE.

590.2 77.0 Rock core recovery 41% Hydrostatic head, El. 680.0 1 July 1944.

D11

671.7 0.0
670.7 1.0 Sandy TOPSOIL
668.3 3.4 Reddish brown, compact, cohesionless, fine, silty and slightly organic SAND.

Gray, compact, slightly cohesive, coarse to fine, gravelly and silty SAND. (silty TILL)

631.7 40.0

Brown, compact, cohesionless coarse to fine, gravelly SAND.

621.7 50.0

Brown, loose, cohesionless, poorly graded, coarse to fine silty SAND.

608.7 63.0

PORPHYRITIC GRANITE with 3.4' weathered surface

588.7 83.0 Rock core recovery 28% Hydrostatic head El. 679.0, 8 July, 1944. Observation well installed.

D12

669.2 0.0
668.2 1.0 Highly organic TOPSOIL
Dark brown, loose, cohesionless, medium to fine highly organic SAND.

659.2 10.0

Gray, compact, cohesionless, coarse to fine gravelly SAND.

655.8 13.4

Gray, very compact, slightly cohesive gravelly silty SAND. (silty TILL). Nest of cobbles from El. 634.2 to El. 624.2

603.0 66.2
601.2 68.0 Brown, loose, cohesionless, coarse to fine SAND.

PORPHYRITIC GRANITE

588.6 80.6 Rock core recovery 52% Hydrostatic head El. 674.3, 15 July 1944.

D13

683.1 0.0
681.9 1.2 Highly organic TOPSOIL

Brown, loose, cohesionless, fine silty SAND.

659.5 23.6

Gray, compact, slightly cohesive gravelly, silty SAND. (silty TILL)

611.6 71.5

PORPHYRITIC GRANITE

596.5 86.6 Rock core recovery 31%

D14

667.6 0.0
666.6 1.0 Highly organic TOPSOIL
Brown, semi-compact cohesionless, uniform, medium to fine SAND

663.0 4.6

Gray, compact, slightly cohesive, gravelly SILT.

652.6 15.0

Brown, semi-compact slightly cohesive gravelly silty SAND. (sandy TILL)

647.6 20.0

Gray, compact, slightly cohesive, gravelly, silty SAND. (sandy TILL)

611.6 56.0

Disintegrated ROCK.

593.2 74.4 No rock core recovered.

D15

681.4 0.0
680.6 0.8 TOPSOIL
680.4 1.0 (Groundwater, 24 July 1944)
678.4 3.0 Brown-gray, loose, cohesionless, fine, silty, organic stained SAND.

671.4 10.0

Brown-gray, compact, slightly cohesive SILT

Gray, very compact slightly cohesive, poorly graded, gravelly, silty SAND (silty TILL).

643.4 38.0

Gray, loose, cohesionless, coarse to fine SAND.

603.8 77.6

PORPHYRITIC GRANITE with shallow weathered surface.

588.4 93.0 Rock core recovery 17%

General Note

Figures at left of log are elevations in feet above M.S.L. Figures at right are depths below ground surface at time of exploration.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

NO.	CHARACTER	DATE	APPR'D
REVISIONS			

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
RECORD OF FOUNDATION EXPLORATION NO. I.

IN SHEETS SHEET NO. SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: Chief, Engineering Division

APPROVED: Colonel, Corps of Engineers, District Engineer

SUBMITTED: Chief, Design Branch, Sr. Engineer

Engineer — DES. BY, DR. BY, TR. BY, CH. BY

FILE NO. M19-13/26

PLATE II-4

D16

670.2 0.0 (Groundwater, 21 July 1944)
6689 1.3 TOPSOIL
Dark brown, compact cohesionless, medium to fine, silty, organic, stained, SAND.
663.2 7.0
Gray, compact, cohesionless, gravelly, silty SAND.
638.6 11.6
Gray, loose, cohesionless, uniform, fine silty SAND.
651.4 18.8
Gray, semi-compact, cohesionless, gravelly silty SAND. (sandy TILL)
646.1 24.1
Gray, compact, slightly cohesive gravelly silty SAND. (silty TILL)
640.2 30.0
PORPHYRITIC GRANITE with shallow weathered surface.
625.2 45.0
Rock core recovery 75%

D17

661.7 0.0
680.7 1.0 TOPSOIL
Red brown, compact, cohesionless, coarse to fine gravelly, organic, stained SAND.
672.2 9.5
Gray, compact, slightly cohesive, fine, silty SAND.
647.7 14.0
Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
634.7 27.0
Gray, compact, slightly cohesive, medium to fine silty SAND.
643.7 38.0
Gray, compact, cohesionless, coarse to fine gravelly SAND. Boulder drilled from El. 638.1 to El. 636.1.
622.7 59.0
Gray, compact, cohesionless, gravelly, silty SAND.
614.7 67.0
612.2 69.5 Gray, compact, slightly cohesive gravelly, silty SAND. (silty TILL)
608.9 72.8
Brown, compact, slightly cohesive, uniform, fine silty SAND.
Weathered ROCK
593.7 88.0
No rock core recovered.

D18

671.9 0.0
Brown, loose, cohesionless, medium to fine, silty SAND.
662.9 9.0
Gray, compact, cohesionless, gravelly, silty SAND.
657.9 14.0
Gray, compact, slightly cohesive, gravelly silty SAND. (silty TILL).
641.9 30.0
Gray, loose, cohesionless, fine SAND and SILT.
629.9 42.0
Brown, compact, cohesionless coarse to fine gravelly SAND.
612.1 59.8
Brown, compact, slightly cohesive, gravelly, silty SAND. (silty TILL) Boulder drilled from El. 607.1 to El. 606.7
599.1 72.8 Artesian flow encountered at El. 610.7
Weathered ROCK
583.9 88.0
No rock core recovered. Observation well installed.

D19

677.2 0.0
676.4 0.8 Organic matter and TOPSOIL
671.6 5.6 Brown, compact, cohesionless, coarse to fine, silty SAND.
Brown-gray, compact, cohesionless, gravelly SAND.
662.7 14.5
Gray, compact, slightly cohesive, coarse to fine, gravelly, silty SAND. (silty TILL).
641.2 36.0
Gray, compact, slightly cohesive fine sandy SILT.
636.2 41.0
Gray, compact, slightly cohesive, coarse to fine, gravelly, silty SAND. (silty TILL).
633.1 44.1
627.8 49.4 Brown, very compact, cohesionless, coarse to
627.2 50.0 medium, gravelly SAND.
622.2 55.0 Gray, compact, slightly cohesive, fine sandy SILT.
618.0 59.2 Gray, compact, slightly cohesive, coarse to fine, gravelly, silty SAND. (silty TILL).
Gray, compact, slightly cohesive, coarse to fine, sandy SILT with decomposed rock fragments.
603.0 74.2 PORPHYRITIC GRANITE
Rock core recovery 7%

D20

673.9 0.0
672.9 1.0 TOPSOIL
672.6 1.3 Brown, loose, cohesionless, uniform, fine, silty SAND.
Groundwater, 1 Aug. 1944.
664.7 9.2
Light brown, very compact, cohesionless, gravelly
658.6 15.3 SAND.
Gray, compact, slightly cohesive, coarse to fine, gravelly, silty SAND. (silty TILL). Boulder drilled from El. 615.9 to El. 610.9.
606.1 67.8
Gray, loose, cohesionless, medium to fine, gravelly, silty SAND.
597.5 76.4 Artesian flow encountered below El. 606.1.
Weathered ROCK
582.5 91.4
No rock core recovered.

D21

678.7 0.0
678.0 0.7 TOPSOIL
Red-brown, compact, cohesionless, poorly graded, gravelly SAND with few cobbles.
672.1 6.6
Brown, compact, cohesionless, coarse to fine, gravelly, silty SAND. (sandy TILL).
644.2 34.5
Gray-brown, compact, cohesionless, medium to
638.9 39.8 fine gravelly, silty SAND.
Brown-gray, compact, cohesionless, coarse to fine, gravelly, silty SAND.
629.5 49.2 (sandy TILL).
PORPHYRITIC GRANITE
614.5 64.2
Rock core recovery 40%.

D22

698.0 0.0
697.5 0.5 Organic matter and TOPSOIL
Red-brown, loose, cohesionless, fine, organic stained SAND.
691.0 7.0
685.0 13.0
Brown, compact, slightly cohesive, coarse to fine, gravelly, silty SAND. (silty TILL). Groundwater, 7 Aug. 1944.
667.3 30.7
Brown, loose, cohesionless, uniform, fine, silty SAND.
653.5 44.5
Gray, compact, cohesionless, fine, silty SAND.
638.0 60.0
Brown, very compact, slightly cohesive, coarse to fine, gravelly, silty SAND.
629.5 68.5 (silty TILL).
625.2 72.8 Brown-gray, very compact, cohesionless, coarse to fine, silty SAND.
PORPHYRITIC GRANITE
610.2 87.8
Rock core recovery 63%

D23

670.5 0.0
668.5 2.0 Gravel road FILL and boulders
667.7 2.8 Organic matter and TOPSOIL
Brown, loose, cohesionless, gravelly SAND with layers of silty SAND.
653.0 17.5
Brown-gray, very compact, slightly cohesive, coarse to fine, gravelly, silty SAND. (silty TILL). Boulder drilled from El. 646.3 to El. 643.7.
613.1 57.4
PORPHYRITIC GRANITE
598.1 72.4
Rock core recovery 95%

D24

694.0 0.0
Brown, loose, cohesionless, gravelly SAND.
682.0 12.0
Gray, loose, cohesionless, fine sandy SILT.
675.0 19.0
Brown-gray, compact, cohesionless, coarse to fine, gravelly, silty SAND. (sandy TILL).
646.9 47.7
Gray, compact, cohesionless, fine, sandy SILT.
610.0 84.0
Gray, compact, cohesionless, coarse to fine silty SAND.
600.0 94.0
PORPHYRITIC GRANITE with shallow weathered surface.
590.1 103.9
Rock core recovery 20%

D25

668.7 0.0
667.3 1.4 TOPSOIL
Brown, loose, cohesionless, uniform, medium to fine
660.5 8.2 SAND.
Brown, compact, cohesionless, coarse to fine, gravelly,
654.5 14.2 silty SAND.
Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
639.4 29.3
PORPHYRITIC GRANITE with shallow weathered surface
624.7 44.0
Rock core recovery 74%

D26

667.7 0.0
666.7 1.0 Gravel road FILL.
665.9 1.8 TOPSOIL
Brown, loose, cohesionless, fine, silty, organic, stained
658.9 8.8 SAND.
Brown-gray, compact, cohesionless, gravelly, silty, SAND. (sandy TILL).
651.2 16.5 Gray, compact, cohesionless,
649.0 18.7 coarse to fine, silty SAND.
646.3 21.4 Gray, very compact, cohesionless, fine, sandy SILT
Gray, very compact, cohesionless, coarse to fine, gravelly SAND.
636.5 31.2
PORPHYRITIC GRANITE
618.4 49.3
Rock core recovery 16%
Observation well installed.

D27

678.2 0.0
677.2 1.0 TOPSOIL
Brown, loose, cohesionless, coarse to fine, gravelly, silty,
670.2 8.0 organic, stained SAND.
Gray, semi-compact, cohesionless, fine, silty SAND.
656.6 21.6
Grey-brown, loose, cohesionless, gravelly SAND.
653.2 25.0
Brown, semi-compact, cohesionless, gravelly,
649.2 29.0 silty SAND.
Gray, compact, cohesionless, uniform, fine, silty SAND.
619.9 58.3
Gray, very compact, cohesionless, medium to fine sandy SILT.
602.2 76.0

D28

668.9 0.0
668.1 0.8 TOPSOIL
Brown, loose, cohesionless, medium to fine, organic,
661.9 7.0 stained SAND.
657.5 11.4 Gray, compact, cohesionless, coarse to fine, gravelly SAND.
Gray, compact, cohesionless, SILT.
649.9 19.0
647.7 21.2 Brown, very compact, cohesionless, gravelly, silty SAND. (sandy TILL).
Gray, very compact, slightly cohesive, gravelly silty SAND. (silty TILL).
634.2 34.7
PORPHYRITIC GRANITE
619.2 49.7
Rock core recovery 52%

D29

679.8 0.0
Brown, gray, loose, cohesionless, medium to fine SAND.
637.3 42.5
Brown, compact, cohesionless, coarse to fine, gravelly, silty SAND.
615.8 64.0
Gray, compact, cohesionless, medium to fine SILT.
591.3 88.5

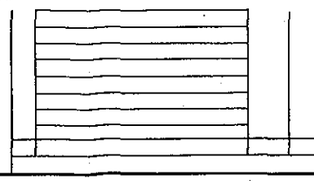
D30

703.1 0.0
702.3 0.8 TOPSOIL
Brown-gray, compact, cohesionless, medium to fine SAND.
693.1 10.0
Gray, compact, cohesionless, medium to fine, silty SAND.
657.9 45.8
Gray, very compact, slightly cohesive, medium to fine SILT.

NOTE

See General Note on Plate II-4.

622.1 81.0



MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
RECORD OF FOUNDATION EXPLORATION NO. 2

IN SHEETS SHEET NO. SCALE AS SHOWN

U.S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: [signature] CHIEF ENGINEERING DIVISION

APPROVED: [signature] COLONEL, CORPS OF ENGINEERS DISTRICT ENGINEER

SUBMITTED: [signature] CHIEF DESIGN BRANCH SR. ENGINEER

W.L. Shannon ENGINEER

FILE NO. M19-13/27

PLATE II-5

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

D31

668.1	0.0	
665.1	3.0	TOPSOIL Red-brown, loose, cohesionless, medium to fine, silty SAND.
659.1	9.0	
		Gray, compact, cohesionless, gravelly, silty SAND.
647.4	20.7	
		Gray, very compact, slightly cohesive, gravelly, silty SAND. (silty TILL). Boulder drilled from El.635.1 to El.633.1
633.1	35.0	
		PORPHYRITIC GRANITE
625.1	43.0	
		Rock core recovery 31%

D32

668.1	0.0	
667.0	1.1	TOPSOIL
665.0	3.1	Brown, loose, cohesionless, medium to fine, silty SAND.
		Brown, slightly compact, cohesionless, gravelly, silty SAND.
654.9	11.2	
		Gray, compact, slightly cohesive, gravelly, silty SAND with streaks of fine silty sand (silty TILL)
643.9	24.8	
		Gray, compact, cohesionless, gravelly, silty SAND.
630.7	37.4	
629.9	38.2	Gray, compact, cohesionless, gravelly, silty SAND. (sandy TILL).
		PORPHYRITIC GRANITE with 0.7' weathered surface
610.8	57.3	
		Rock core recovery 4%

D33

692.7	0.0	
692.2	0.5	TOPSOIL
		Gray-brown, loose, cohesionless, coarse to fine SAND.
678.1	14.6	
672.7	20.0	Gray, loose, slightly cohesive, stratified sandy SILT with occasional gravel.
		Gray-brown, loose, cohesionless, coarse to fine, gravelly, silty SAND.
659.7	33.0	
		Gray, compact, cohesionless, medium to fine, silty SAND.
634.7	58.0	
		Gray, very compact, cohesionless, coarse to fine, gravelly, silty SAND.
612.7	80.0	
607.3	85.4	Gray, compact, cohesionless, medium to fine, silty SAND.
605.7	87.0	Dark-gray, compact, slightly cohesive, medium to fine SILT.

D34

718.2	0.0	
717.8	0.4	TOPSOIL
		Gray, loose, cohesionless, gravelly SAND.
709.2	9.0	
		Gray, loose, cohesionless, medium to fine, silty SAND.
662.9	55.9	
		Brown, compact, cohesionless, gravelly, silty SAND.
652.2	66.0	
		Gray, compact, cohesionless, fine, silty SAND.
633.2	85.0	
		Gray, compact, slightly cohesive, medium to fine SILT.
624.3	93.9	

D35

684.3	0.0	
683.3	1.0	TOPSOIL
		Brown, loose, cohesionless, medium to fine SAND.
675.3	9.0	
		Gray, compact, cohesionless, gravelly SAND.
668.7	15.6	
665.3	19.0	Gray, compact, cohesionless, gravelly, silty SAND.
		Gray, very compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
640.0	44.3	
		PORPHYRITIC GRANITE Badly weathered to El.636.0
625.4	58.9	
		Rock core recovery 52%

D36

673.3	0.0	
672.9	0.4	TOPSOIL
		Brown, compact, cohesionless, gravelly SAND.
665.3	8.0	
		Brown-gray, compact, cohesionless, gravelly, silty SAND. (sandy TILL).
656.3	17.0	
		Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
640.0	33.3	
		Brown, very compact, cohesionless, gravelly, silty SAND. (sandy TILL).
623.3	50.0	
		Rock core recovery 80% (Artesian flow encountered at El.609.0)
608.3	65.0	

D37

671.6	0.0	
669.1	2.5	TOPSOIL
667.8	3.8	Red-brown, loose, cohesionless, medium to fine, organic, stained SAND.
662.1	9.5	(Groundwater, 30 Aug. 1944)
		Gray, compact, cohesionless, gravelly SAND cobbles.
656.6	15.0	Brown, compact, cohesionless, gravelly, silty SAND. (sandy TILL).
		Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
641.6	30.0	

D38

672.8	0.0	
672.0	0.8	TOPSOIL
		Red, loose, cohesionless, gravelly SAND and organic matter.
666.3	6.5	
		Brown, compact, cohesionless, gravelly, silty SAND. (sandy TILL).
654.8	18.0	
		Gray, compact, slightly cohesive, gravelly, silty SAND. (Variable sandy and silty TILL).
641.4	31.4	

D39

673.8	0.0	
673.0	0.8	TOPSOIL
		Brown, loose, cohesionless, medium to fine, silty SAND.
668.0	5.8	
		Gray, compact, cohesionless, fine, silty SAND.
654.8	19.0	
651.8	22.0	Brown, compact, cohesionless, gravelly, silty SAND. (sandy TILL).
		Gray, very compact, slightly cohesive, gravelly, silty SAND with occasional lenses of fine silty sand (silty TILL). Boulder drilled from El.635.6 to El.634.3
606.3	67.5	
		Decomposed ROCK
590.9	82.9	
		No rock core recovered.

D40

676.5	0.0	
675.5	1.0	TOPSOIL and boulders
		Brown, gray, loose to compact, gravelly SAND with organic matter and cobbles.
670.5	6.0	
		Brown, gray, compact, cohesionless, gravelly, silty SAND - cobbles. (sandy TILL).
658.5	18.0	
		Gray, very compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
647.5	29.0	

D41

672.3	0.0	
671.5	0.8	ORGANIC MATTER
669.3	2.5	Red-brown, loose, cohesionless, medium to fine, silty, organic, stained SAND.
668.1	4.2	
		Gray, brown, loose, cohesionless, gravelly, silty SAND.
		Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL)
652.3	20.0	

D42

679.2	0.0	
678.6	0.6	ORGANIC MATTER
		Brown, loose, cohesionless, gravelly SAND and organic matter. Boulders encountered.
671.7	7.5	
667.4	11.8	Brown, compact, cohesionless, gravelly and silty SAND. (sandy TILL).
		Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
648.2	31.0	

D43

677.6	0.0	
676.0	1.6	TOPSOIL
		Gray, loose, cohesionless, medium to fine silty SAND.
664.3	13.3	
		Gray, compact, slightly cohesive, gravelly, silty SAND - cobbles. (silty TILL).
643.1	34.5	

D44

677.8	0.0	
676.8	1.0	Highly organic TOPSOIL
		Brown, loose, cohesionless, medium to fine, silty SAND. Boulder encountered from El.663.8 to El.662.8.
661.8	16.0	
		Brown, very compact, cohesionless, gravelly, silty SAND. (sandy TILL)
656.3	21.5	
		Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL)
641.8	36.0	

D45

673.3	0.0	
672.5	0.8	Highly organic TOPSOIL
670.6	2.7	Brown, loose, cohesionless, medium to fine, organic, stained SAND.
665.5	7.8	
		Brown, gray, compact, cohesionless, gravelly SAND. Boulders encountered.
		Brown, gray, compact, slightly cohesive, gravelly SAND with lenses of fine, silty sand. (silty TILL).
643.3	30.0	

NOTE

See General Note on Plate II-4

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

RECORD OF FOUNDATION EXPLORATION NO. 3.

IN SHEETS — SHEET NO. — SCALE AS SHOWN

U.S. ENGINEER OFFICE, BOSTON, MASS. — 18 APRIL 1945

APPROVAL RECOMMENDED
Chief Engineering Division

APPROVED:
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

SUBMITTED:
Chief Design Branch or Engineer
W. L. Cummin
Engineer

FILE NO. M 19-13/28

PLATE II-6

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

D46

724.3	0.0
723.3	1.0 Highly organic TOPSOIL
	Gray, loose, cohesionless, coarse to fine, silty SAND.
713.3	11.0
	Gray, compact, cohesionless, gravelly SAND.
708.3	16.0
706.8	17.5 Red-brown, compact,
	cohesionless, coarse to fine,
703.2	21.1 silty SAND.
	Brown, compact, cohesionless, gravelly, silty SAND.(sandy TILL)
	Gray, compact, slightly cohesive, gravelly, silty SAND with cobbles (sandy TILL)
674.3	50.0

D47

687.8	0.0
687.6	0.2 Highly organic TOPSOIL
	Brown-gray, semi-compact, medium to fine, silty SAND.
667.2	20.6
	Gray, compact, slightly cohesive, gravelly, silty SAND.(silty TILL).
643.8	44.0
	PORPHYRITIC GRANITE
628.8	59.0
	Rock core recovery 40%

D48

668.0	0.0
666.8	1.2 Highly organic TOPSOIL
663.2	4.8 Brown, loose, cohesionless,
	coarse to fine SAND.
659.7	8.3 Gray, semi-compact,
	cohesionless, medium to
	fine, silty SAND.
	Gray, compact, cohesionless, gravelly, silty SAND.
650.0	18.0
	Gray, compact, slightly cohesive, gravelly, silty SAND.(silty TILL).
618.2	49.8

D49

681.6	0.0
681.1	0.5 Highly organic TOPSOIL
678.3	3.3 Brown, loose, cohesionless,
676.8	4.8 medium to fine, organic,
	stained SAND.
	Gray, loose, cohesionless,
669.5	12.1 coarse to fine, silty SAND.
	Brown, gray, compact, cohesionless, gravelly, silty SAND with few cobbles.
	Gray, compact, slightly cohesive, gravelly, silty SAND.(silty TILL) Boulder drilled from El. 660.5 to El. 655.5
646.4	35.2
	Gray, compact, cohesionless, gravelly, silty SAND.
639.4	42.2
	Observation well installed.

D50

666.0	0.0
665.6	0.4 Highly organic TOPSOIL
661.9	4.1 Brown, loose, cohesionless,
	medium to fine, organic, stained SAND.
	Brown-gray, loose, cohesionless, coarse to fine, silty SAND.
644.8	21.2
	Gray, very compact, slightly cohesive, gravelly, silty SAND.(silty TILL).
612.9	53.1

D51

697.9	0.0
697.5	0.4 Organic TOPSOIL
694.9	3.0 Brown, loose, cohesionless,
	medium to fine SAND with
689.9	8.0 little organic matter.
	Brown, loose, cohesionless,
684.9	13.0 medium to fine, silty SAND.
	Gray, compact, cohesionless, gravelly SAND.
	Gray, compact, slightly cohesive, gravelly, silty SAND.(silty TILL).
663.9	34.0

D52

680.0	0.0
679.4	0.6 Highly organic TOPSOIL
674.5	5.5 Brown, loose, cohesionless,
	gravelly, silty, organic,
	stained SAND.
670.2	9.8 Boulders encountered.
	Gray, loose, cohesionless, gravelly SAND with cobbles and boulders.
	Brown-gray, compact,
659.5	20.5 cohesionless, gravelly,
	silty SAND.(sandy TILL). Boulder drilled from El. 660.8 to El. 660.0
	Gray, compact, slightly cohesive, gravelly, silty SAND with variable layers of silty sand and silt. (silty TILL).
634.7	45.5
630.8	49.2 Decomposed ROCK

D53

670.1	0.0
669.0	1.1 Highly organic TOPSOIL
666.7	3.4 Brown, compact, cohesionless,
	gravelly, silty SAND
	(sandy TILL).
	Gray, compact, slightly cohesive gravelly silty SAND.(silty TILL)
651.3	18.8

D54

681.6	0.0
681.1	0.5 Highly organic TOPSOIL
	Brown, loose, cohesionless,
674.9	6.7 medium to fine, organic,
	stained SAND.
	Brown-gray, compact, cohesionless, variable, gravelly, silty SAND.
663.6	18.0
	Gray, compact, slightly cohesive, gravelly, silty SAND with layers of silty sand. (silty TILL).
651.1	30.5

D55

697.3	0.0
696.3	1.0 Organic TOPSOIL
	Brown, loose, cohesionless,
690.8	6.5 medium to fine SAND.
	Gray, loose, cohesionless, medium to fine, silty SAND.
682.0	15.3
	Brown, loose, cohesionless, coarse to fine SAND.
667.3	30.0

D56

709.0	0.0
707.0	2.0 Organic TOPSOIL
	Brown, loose, cohesionless,
	medium to fine, organic,
701.4	7.6 stained SAND.
698.1	10.9 Brown, loose, cohesionless, gravelly SAND.
	Gray, loose, cohesionless, medium to fine, silty SAND.
687.2	21.8
	Gray, compact, cohesionless, gravelly, silty SAND. Boulder drilled from
679.5	29.5 El. 681.6 to El. 679.7.

D57

700.3	0.0
700.0	0.3 TOPSOIL
	Gray, loose, cohesionless, gravelly, silty SAND.
690.3	10.0
	Brown, compact, cohesionless, gravelly, silty SAND. (sandy TILL).
654.1	46.2
	Gray, compact, cohesionless,
649.7	50.6 medium to fine, silty SAND.
	Observation well installed.

D58

671.8	0.0
670.6	1.2 TOPSOIL
	Brown, loose, cohesionless, medium to fine SAND.
655.8	16.0
	Brown, gray, loose, cohesionless, variable, gravelly SAND with lenses of fine, silty sand
635.8	36.0
	Brown, gray, semi-compact, cohesionless, medium to fine, silty SAND.
621.8	50.0

D59

658.2	0.0
	Brown, loose, cohesionless, gravelly SAND.
652.4	5.8 Boulders and cobbles encountered.
	Gray, compact, slightly cohesive, gravelly, silty SAND.(silty TILL).
609.2	49.0
608.2	50.0 Decomposed ROCK

D60

706.4	0.0
703.9	0.5 Organic TOPSOIL
702.3	4.1 Brown, loose, cohesionless,
	medium to fine, organic,
	stained SAND.
	Gray, loose, cohesionless, gravelly SAND with few cobbles.
676.6	29.8
	Gray, very compact, cohesionless, gravelly, silty SAND.
667.0	39.4
	Gray, very compact, cohesionless, gravelly, silty SAND.(sandy TILL).
656.4	50.0

See General Note on PLATE II-4

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

RECORD OF FOUNDATION EXPLORATION NO. 4.

IN SHEETS SHEET NO. SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: John Sutton, CHIEF ENGINEERING DIVISION

APPROVED: Jonny B. Pratt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN BRANCH OR ENGINEER

ENGINEER

FILE NO. M 19-13/29

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

PLATE II-7

D61

660.2 0.0
Brown, gray, loose, cohesionless, sandy GRAVEL.
653.2 7.0
Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL).
643.2 17.0

D62

697.1 0.0
696.7 0.4 TOPSOIL
Brown, loose, cohesionless, silty, organic, stained SAND.
685.9 11.2
Gray, very compact, cohesionless, gravelly SAND.
681.1 16.0
Brown, very compact, cohesionless, gravelly, silty SAND.
675.6 21.5
Gray, very compact, cohesionless, gravelly, silty SAND. (sandy TILL) cobbles.
666.7 30.4

D63

669.7 0.0
668.7 1.0 TOPSOIL
666.7 3.0 Brown, loose, cohesionless, medium to fine, silty SAND.
Gray, loose, cohesionless, gravelly, silty SAND.
654.3 15.4
Gray, very compact, slightly cohesive, gravelly, silty SAND. (silty TILL)
643.7 26.0

D64

726.8 0.0
726.5 0.3 TOPSOIL
721.5 5.3 Brown-gray, loose, cohesionless, variable, gravelly, silty SAND.
Gray, loose, cohesionless, medium to fine, silty SAND.
692.0 34.8
Gray, compact, cohesionless, gravelly, silty SAND.
678.6 48.2
674.6 52.2 Gray, compact, cohesionless, fine, silty SAND.

D65

671.0 0.0
670.0 1.0 TOPSOIL
Brown-gray, loose, cohesionless, medium to fine, silty SAND.
648.0 23.0
Gray, compact, slightly cohesive, gravelly, silty SAND. (silty TILL)
638.0 33.0

D66

676.7 0.0
675.7 1.0 Highly organic TOPSOIL
Light brown, loose cohesionless, medium to fine silty SAND.
666.1 10.0
Brown, compact, cohesionless gravelly, silty SAND (sandy TILL) Boulders encountered at El. 659.7
652.1 24.6
Gray, compact, slightly cohesive, gravelly, silty SAND with lenses of laminated silt (silty TILL)
625.4 45.0
Decomposed ROCK
611.9 64.8
Brown, compact, cohesionless, gravelly and silty SAND
611.5 65.2 Decomposed ROCK
(No recovery)
608.5 73.2
PORPHYRITIC GRANITE badly weathered and fractured
(Hydrostatic head at ground level, 12 Dec. 1944.
571.5 105.2
Rock core recovery 44.5% 570.8
Observation well installed.

D67

673.4 0.0
672.0 1.4 Organic TOPSOIL
Brown, semi-compact medium to fine SAND.
670.4 3.0 Brown, compact, cohesionless fine silty SAND.
Boulder drilled from
660.4 13.0 El. 665.2 to El. 664.4
Gray, compact, slightly cohesive, gravelly, silty SAND with layers of fine sand (silty TILL)
625.4 46.0
Decomposed ROCK
607.4 66.0
PORPHYRITIC GRANITE badly weathered to El. 587.8
94.6
Rock core recovery 24%
Observation well installed.

T43P

0.0
2.0 TOPSOIL
Compact gravelly SAND with boulders
5.0 Compact SAND and GRAVEL
9.0

T45P

0.0
1.3 Organic TOPSOIL with boulders.
Brown fine SAND and SILT with boulders.
3.0 Brown, compact, cohesionless gravelly, silty SAND (sandy TILL)
5.7 Gray, compact, slightly cohesive, gravelly, silty SAND (silty TILL)
6.0

T56P

0.0
1.2 TOPSOIL
3.0 Brown, compact, fine, silty SAND.
Brown, compact, slightly cohesive, gravelly, silty, SAND containing cobbles and boulders (sandy TILL)
(side of pit shows 6'-0" diameter boulder)
4.5 Blue gray, semi-compact, slightly cohesive, gravelly silty SAND containing cobbles and boulders (silty TILL)
6.0

T57P

0.0
1.0 HUMUS and TOPSOIL.
Brown, compact, silty, SAND (Boulders encountered in sides and bottom of pit
3.0

T58P

0.0
1.0 Yellow-gray, compact uniform SAND
Yellow-brown, compact variable, gravelly SAND containing cobbles (boulder area
3.0 encountered at 3.0')

T59t

0.0
0.6 TOPSOIL.
Yellow-gray, compact, uniform SAND
20.0

T60P

0.0
1.0 Organic TOPSOIL
Very compact GRAVEL with large boulders
12.0
Gray, compact, slightly cohesive, gravelly, silty SAND (silty TILL)
24.0

T1t

0.0
2.0 Brown, sandy TOPSOIL
Light brown, uniform, fine to medium SAND fairly loose.
3.8 dry.

SEISMIC DATA

Seismic Line	Surface Elev.	Approx Rock El.
1	684.7	619.0
2	666.7	585.0
3	668.4	622.0
4	682.0	636.0
5	698.3	629.0
6	671.3	607.0
7	694.6	613.0
8	690.9	570.0
9	731.3	Deeper than 634.0
10	675.7	649.0
11	674.2	651.0
12	724.4	675.0
13	698.0	649.0
14	692.2	644.0
15	675.9	Deeper than 616.0
16	682.5	Deeper than 634.0
17	687.3	647.0
18	674.3	652.0

See General Note on PLATE II-4

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
RECORD OF FOUNDATION EXPLORATION NO. 5

IN SHEETS SHEET NO. SCALE AS SHOWN

U.S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1946

APPROVAL RECOMMENDED: Chief, Engineering Division

APPROVED: Colonel, Corps of Engineers, District Engineer

SUBMITTED: Chief, Design Branch, Engineer

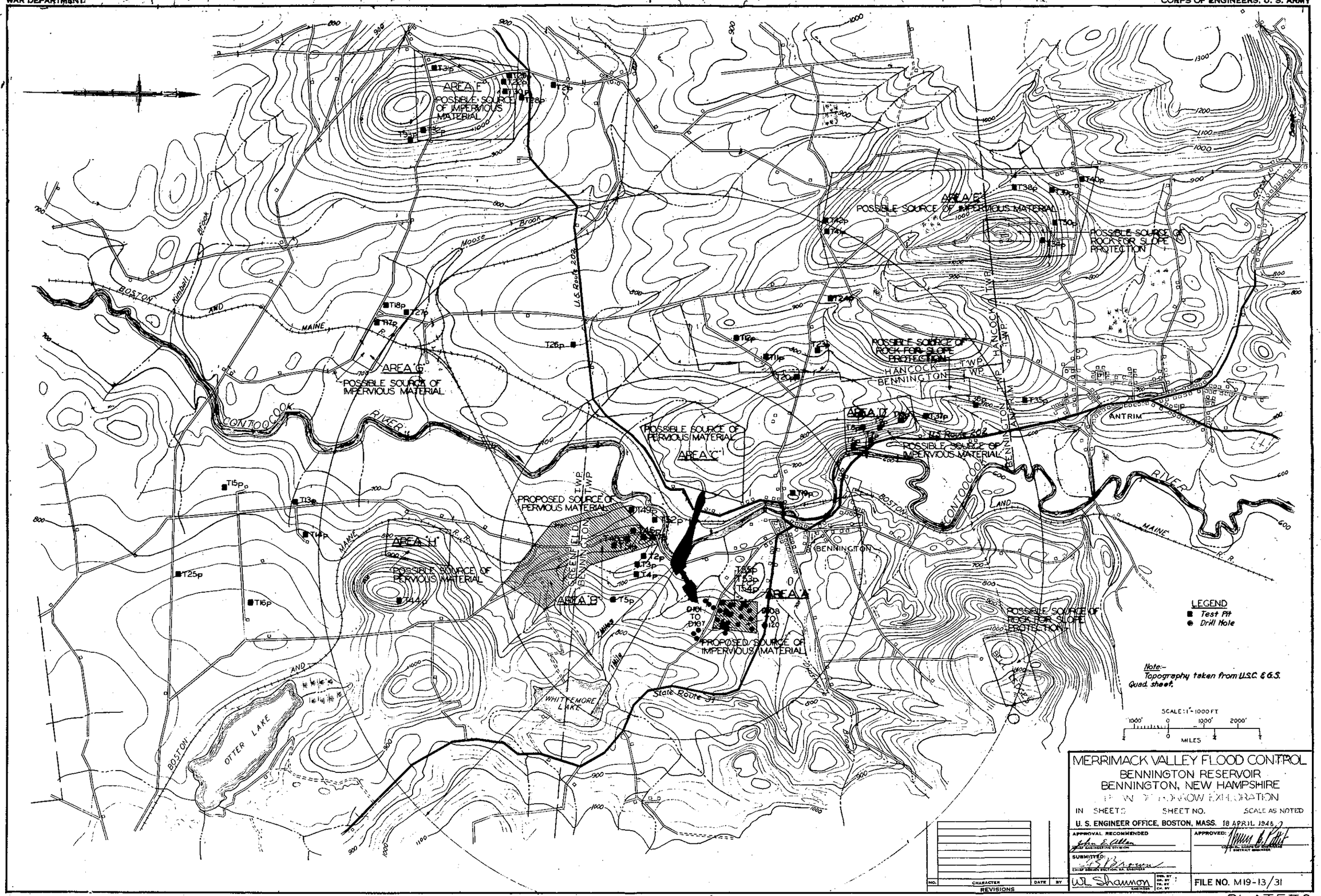
W. L. Shamm, Engineer

FILE NO. M 19-13/30'

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

WAR DEPARTMENT.

CORPS OF ENGINEERS, U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
BENNINGTON, NEW HAMPSHIRE
PLAN OF FOUNDATION EXPLORATION

IN SHEETS | SHEET NO. | SCALE AS NOTED

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: John E. Allen, Chief Engineering Division

SUBMITTED: Chief Design Section, Sr. Engineer

APPROVED: Thomas B. Cutts, Colonel, Corps of Engineers, District Engineer

W. L. Shannon, Engineer — DES. BY | DR. BY | TR. BY | CH. BY

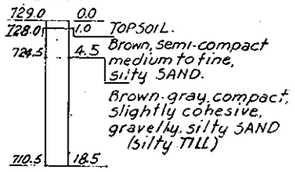
FILE NO. M19-13/31

NO.	CHARACTER	DATE	BY

REVISIONS

PLATE II-9

D-108



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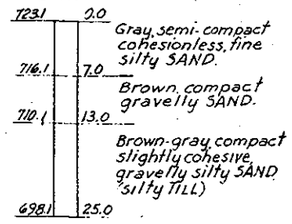

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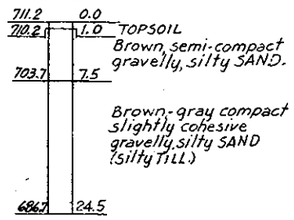

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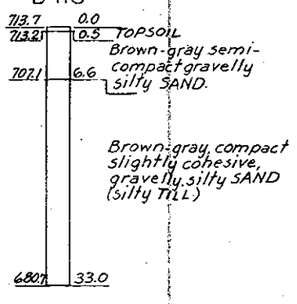

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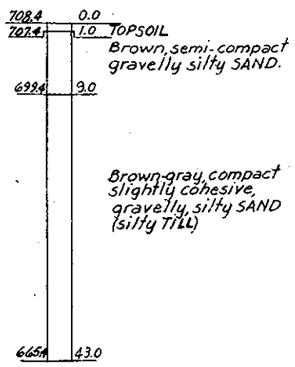

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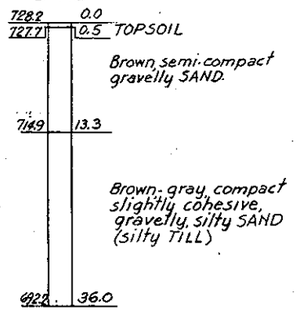

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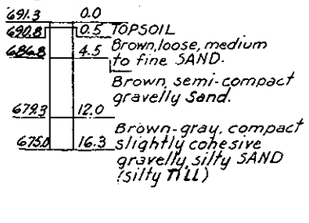

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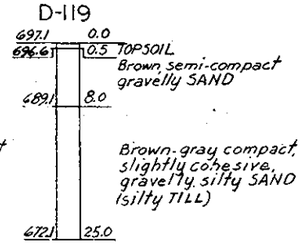
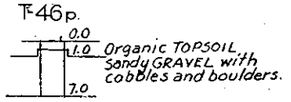


T-46p.



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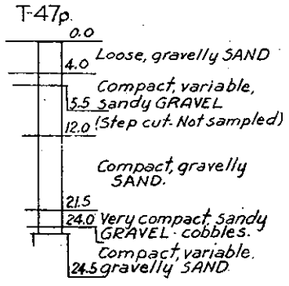

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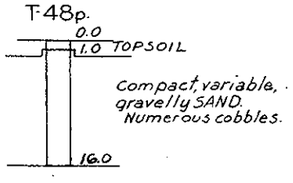

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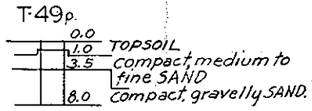

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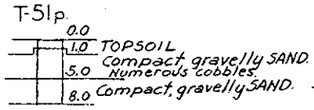
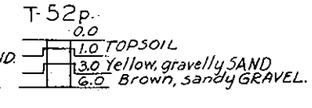


T-52p.



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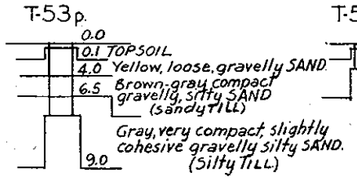

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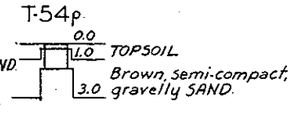

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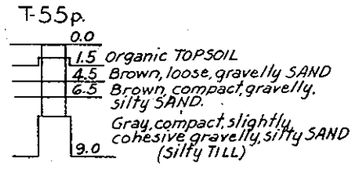


NOTE

This record includes only selected representive explorations.

See General Note on Plate II-4

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

RECORD OF BORROW EXPLORATION

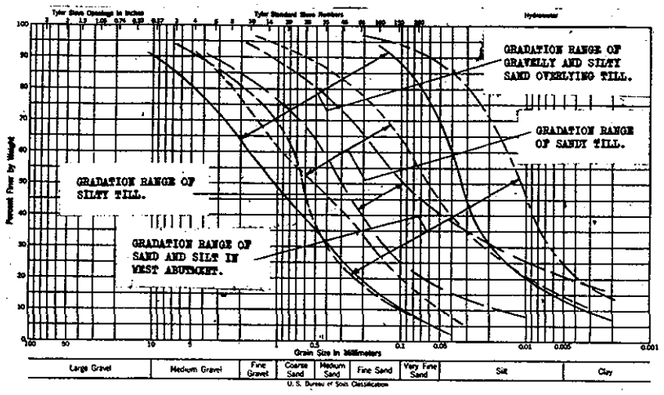
IN SHEETS SHEET NO. SCALE: AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1935

APPROVAL RECOMMENDED: Edgar L. Allen, CHIEF ENGINEERING DIVISION	APPROVED: Thomas J. Cole, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER
SUBMITTED: W. L. Shannon, CHIEF DESIGN BRANCH SR. ENGINEER	
W. L. Shannon, ENGINEER	FILE NO. M 19-13/32.

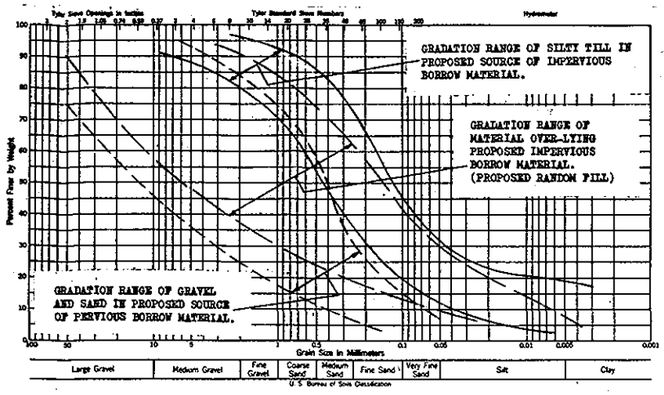
THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

PLATE II-10



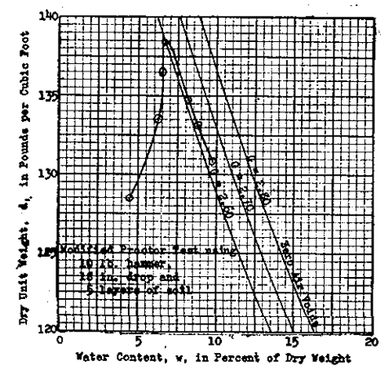
GRADATION OF FOUNDATION MATERIAL

FIG. 1



GRADATION OF BORROW MATERIALS

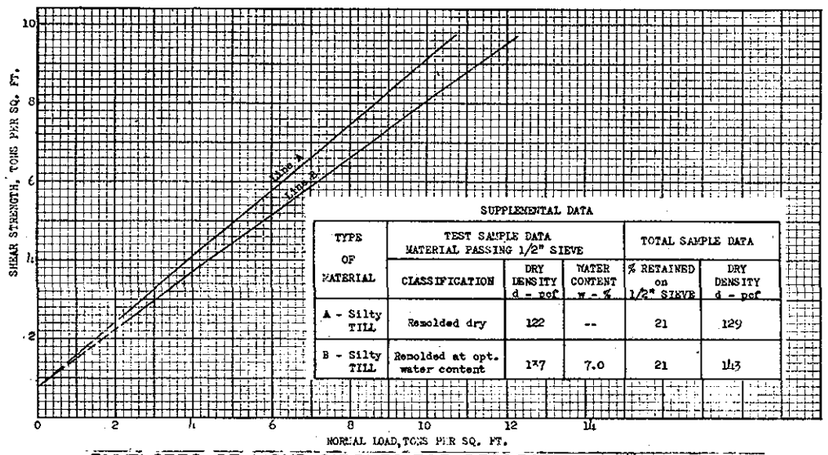
FIG. 2



MAX. DENSITY < #4 MAT'L. = 138.2 p.c.f.
OPT. WATER CONT.<#4 MAT'L. = 6.7 %
COMPUTED MAX. DENSITY OF TOTAL SAMPLE = 142.2 p.c.f.
MAT'L. RETAINED ON #4 SIEVE = 21.4 %
SPECIFIC GRAVITY OF SOIL = 2.71

MOISTURE DENSITY CURVE FOR SILTY TILL

FIG. 3

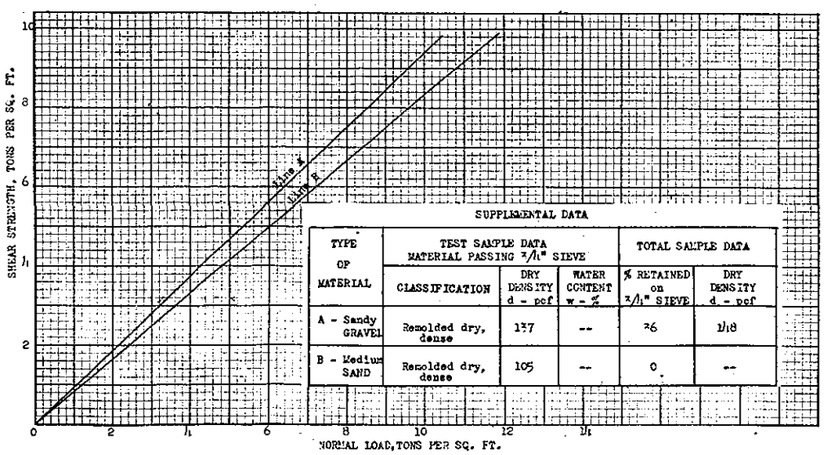


SUPPLEMENTAL DATA

TYPE OF MATERIAL	TEST SAMPLE DATA MATERIAL PASSING 1/2" SIEVE			TOTAL SAMPLE DATA	
	CLASSIFICATION	DRY DENSITY d - pcf	WATER CONTENT w - %	% RETAINED on 1/2" SIEVE	DRY DENSITY d - pcf
A - Silty TILL	Remolded dry	122	--	21	129
B - Silty TILL	Remolded at opt. water content	137	7.0	21	143

ENVELOPES OF MOHR CIRCLES FOR REPRESENTATIVE SAMPLE OF IMPERVIOUS MATERIAL IN CONSOLIDATED QUICK SHEAR TESTS

FIG. 4



SUPPLEMENTAL DATA

TYPE OF MATERIAL	TEST SAMPLE DATA MATERIAL PASSING 3/4" SIEVE			TOTAL SAMPLE DATA	
	CLASSIFICATION	DRY DENSITY d - pcf	WATER CONTENT w - %	% RETAINED on 3/4" SIEVE	DRY DENSITY d - pcf
A - Sandy GRAVEL	Remolded dry, dense	137	--	26	148
B - Medium SAND	Remolded dry, dense	105	--	0	--

ENVELOPES OF MOHR CIRCLES FOR REPRESENTATIVE SAMPLES OF PERVIOUS MATERIAL IN CONSOLIDATED QUICK SHEAR TESTS

FIG. 5

SUMMARY OF PROPERTIES OF MATERIALS ENCOUNTERED

IDENTIFICATION			NATURAL PROPERTIES					REMOLDED PROPERTIES					
								COMPACTION CHARACTERISTICS					
								LABORATORY TEST DATA (a)			TOTAL SAMPLE		
TYPE	OCCURRENCE	CLASSIFICATION	WATER CONTENT w % dry wt.	DRY DENSITY d p.c.f.	VOID RATIO e	PERMEABILITY k 10^{-4} cm/sec	SHEAR STRENGTH ϕ deg.	MIN. DRY DENSITY p.c.f.	MAX. DRY DENSITY p.c.f.	OPT. WATER CONTENT w % dry wt.	MAX. DRY DENSITY p.c.f.	PERMEABILITY k 10^{-4} cm/sec	SHEAR STRENGTH ϕ deg.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Foundation Materials	Unconsolidated Surface Deposits	Variable gravelly and silty SAND	20.1(b) (14) 4.5-31.1	113.6 (19) 91-134	0.50 (17) 0.22-0.85	5.0 (c) est.	35 est.	97.9(d) (3) 88-103	121.3(e) (3) 109-131	----	123.0 (3) 115-131	5.0 est.	35 est.
	Upper zone of TILL	Weathered brown sandy TILL	10.3 (24) 7.1-18.9	128.0 (7) 116-134	0.31 (5) 0.30-0.36	0.1 est.	35 est.	-----	126.7 (2) (f) 125-129	9.0 (2) 8.0-10.0	128.5 (2) 126-131	0.1 est.	35 est.
	Principal TILL body	Gray compact silty TILL	9.4 (31) 5.7-11.6	132.7 (25) 121-144	0.29 (18) 0.22-0.38	0.01 est.	35 est.	-----	135.7 (4) 133-138	7.4 (4) 7.0-8.6	139.5 (4) 137-142	0.01 est.	See Fig. 4
	West Abutment	SAND and SILT	5.4 (6) 3.8-7.3	105.2 (6) 91-134	0.65 (6) 0.27-0.85	1.0 est.	30 est.	103.4 (1)	130.6 (1)	-----	130.6 (1)	1.0 est.	30 est.
Borrow Materials	Impervious Fill	Silty TILL	9.4 (18) 5.7-11.7	130.8 (21) 121-144	0.29 (18) 0.22-0.38	0.01 est.	35 est.	-----	135.7 (4) 133-138 See Fig. 3	7.4 (4) 7.0-8.6	139.5 (4) 137-142	0.01 est.	See Fig. 4
	Random Fill	Silty SAND and sandy TILL	19.3 (12) 9.4-27.2	105.7 (9) 84-131	0.44 (11) 0.23-0.82	1.0 est.	35 est.	-----	122.3 (3) 109-130	10.7 (3) 9.5-12.5	133.9 (3) 115-143	1.0 est.	35 est.
	Pervious Fill	GRAVEL and SAND	3.6 (19) 3.0-6.8	127.5 (23) 98-150	0.47 (6) 0.36-0.56	10-100 est.	35 est.	95.7 (5) 88-105	120.2 (5) 110-124	-----	133.0 (5) 110-154	10-100 est.	See Fig. 5

NOTES

(a) Laboratory test data for material passing No. 4 mesh sieve.
(b) Except where used to number column headings, figures in ( ) under average data indicate number of tests. Figures below ( ) indicate range of test results.
(c) "est." indicates estimated values based on tests on similar materials for other sites.
(d) Minimum density obtained by placing dry material in container without compaction or vibration.
(e) Maximum density obtained by impact compaction in thin layers with complete saturation.
(f) Modified Proctor Test.

FIG. 6

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

SOIL DATA SUMMARY

IN SHEETS SHEET NO. SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: John E. Sullivan, CHIEF ENGINEERING DIVISION

APPROVED: Thomas B. Hall, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTER: CHIEF DESIGN BRANCH, SR. ENGINEER

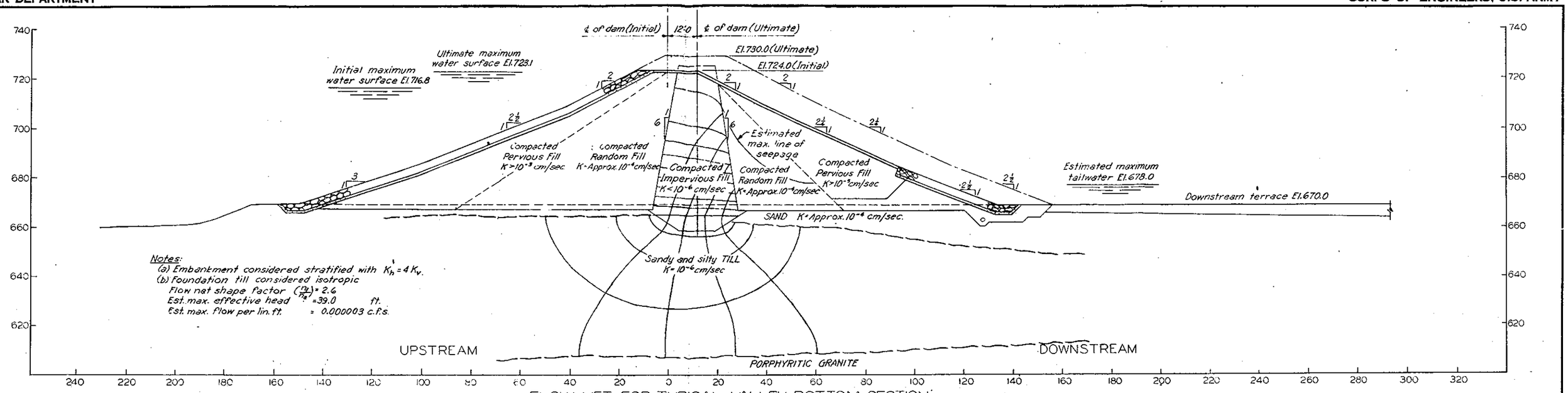
NO.	CHARACTER	DATE	APP'D
	REVISIONS		

DES. BY DR. BY TR. BY CH. BY

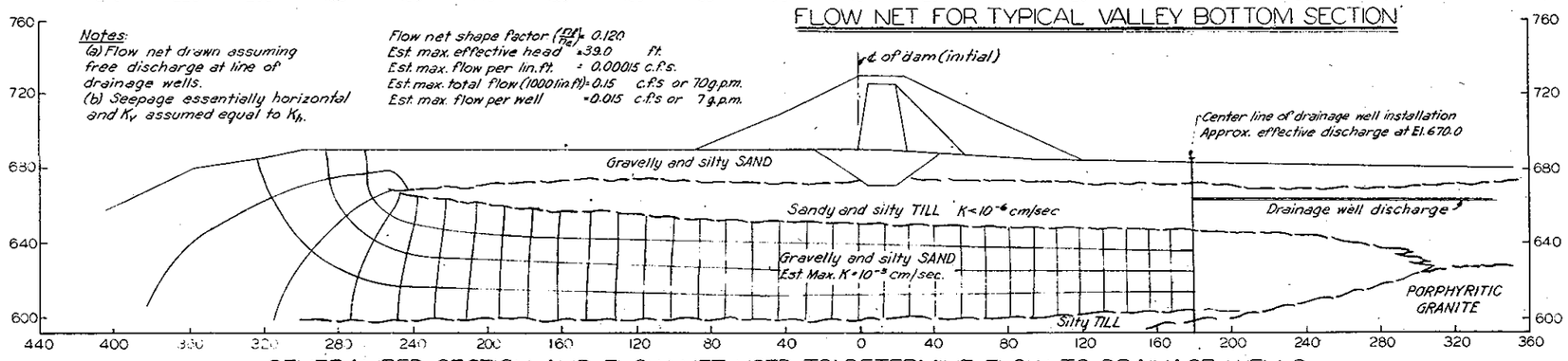
FILE NO. M19-13/33

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

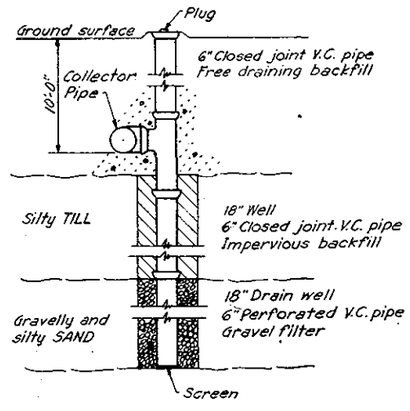
PLATE II-11



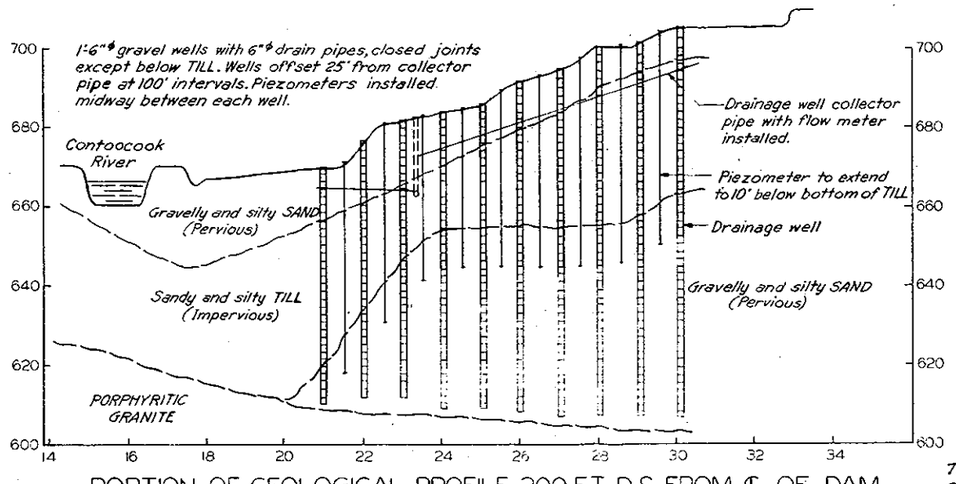
FLOW NET FOR TYPICAL VALLEY BOTTOM SECTION



GENERALIZED SECTION AND FLOW NET USED TO DETERMINE FLOW TO DRAINAGE WELLS



DRAINAGE WELL DETAIL



PORTION OF GEOLOGICAL PROFILE 200 FT. D.S. FROM ℄ OF DAM SHOWING PROPOSED DRAINAGE WELL INSTALLATION

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

NOTE:
Elevations are based on U.S.C. & G.S. Datum of Mean Sea Level.

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

SEEPAGE ANALYSIS

IN SHEETS SHEET NO. SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED:
Chief Engineering Division

APPROVED:
Colonel, Corps of Engineers
District Engineer

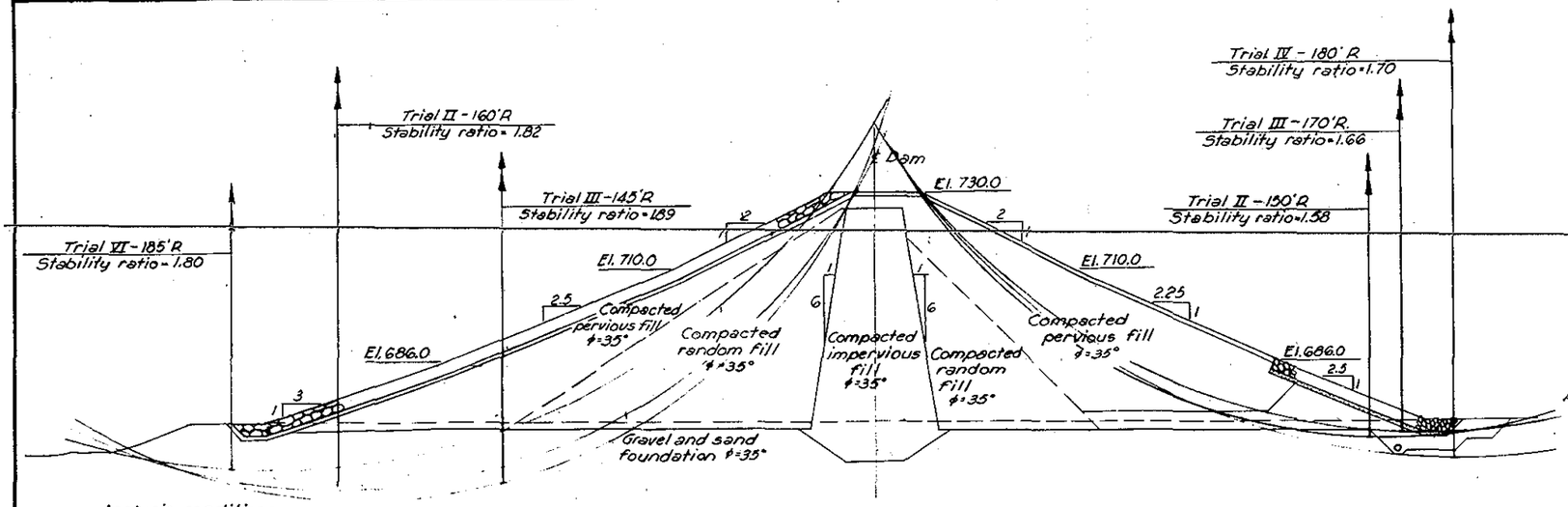
SUBMITTED:
Chief Design Branch, Sr. Engineer

Engineer

DRW. BY
DR. BY
TR. BY
CH. BY

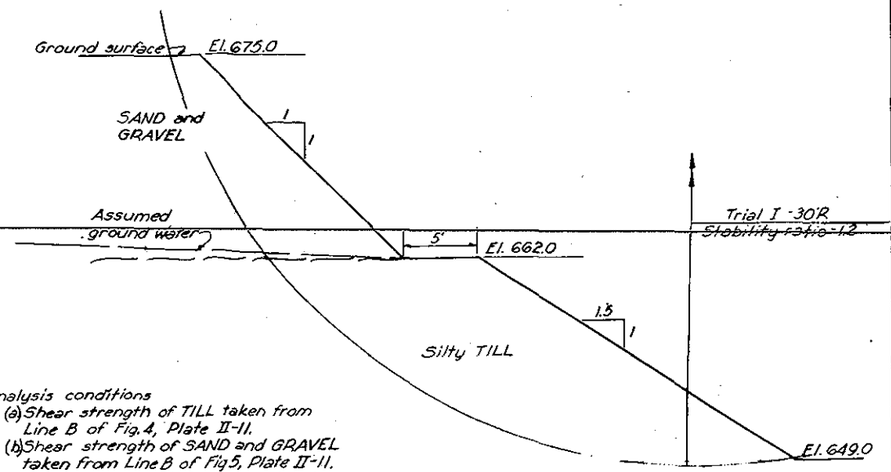
FILE NO. M19-13/34

PLATE II-12



Analysis conditions
(a) Shear strength of all materials represented by an angle of internal friction of 35° for conservative preliminary analysis.
(b) Upstream slope analyzed for case of sudden drawdown of pool from El. 717 to El. 667 with full drainage of compacted pervious and random sections
(c) Downstream slope analyzed for case of sudden drawdown of pool from El. 678 to El. 667 with full drainage of compacted pervious and random sections.

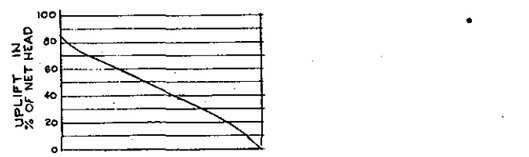
TYPICAL ULTIMATE EMBANKMENT SECTION

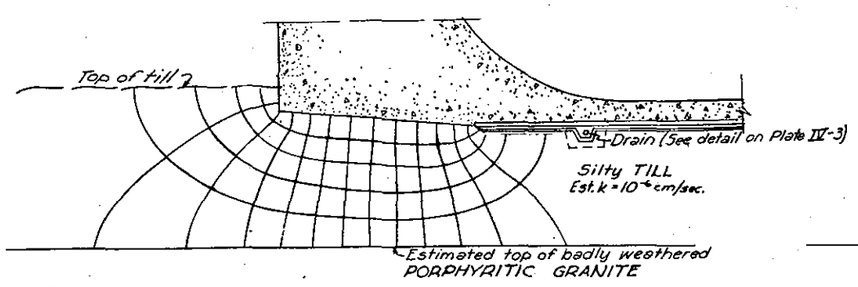


Analysis conditions
(a) Shear strength of TILL taken from Line B of Fig. 4, Plate II-11.
(b) Shear strength of SAND and GRAVEL taken from Line B of Fig 5, Plate II-11.
(c) Slope analyzed for case of full drainage in gravel and no drainage in till.

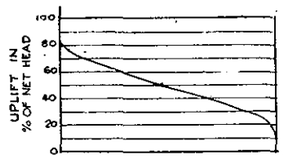
SLOPES FOR SPILLWAY EXCAVATION
Scale: 1"=5'

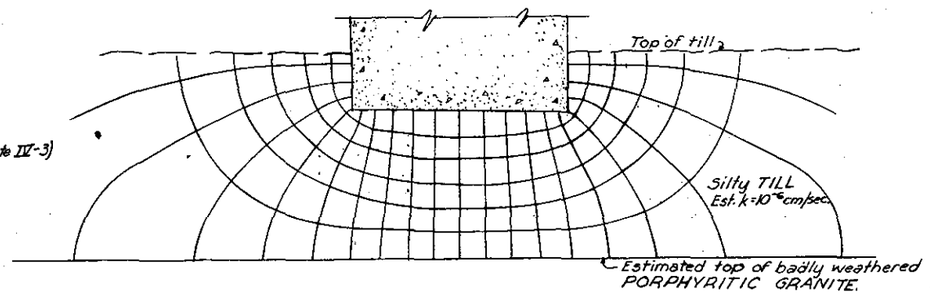
STABILITY ANALYSES





OVERFLOW SECTION





NON OVERFLOW SECTION

FLOW NETS AND UPLIFT DIAGRAMS FOR SPILLWAY

NOTE:
Elevations based on U.S.C.&G.S. Datum of Mean Sea Level.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
STABILITY AND SEEPAGE ANALYSIS
20' 0' 20' 40'
IN SHEETS SHEET NO. SCALE: 1 IN. = 20 FT.
U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1935

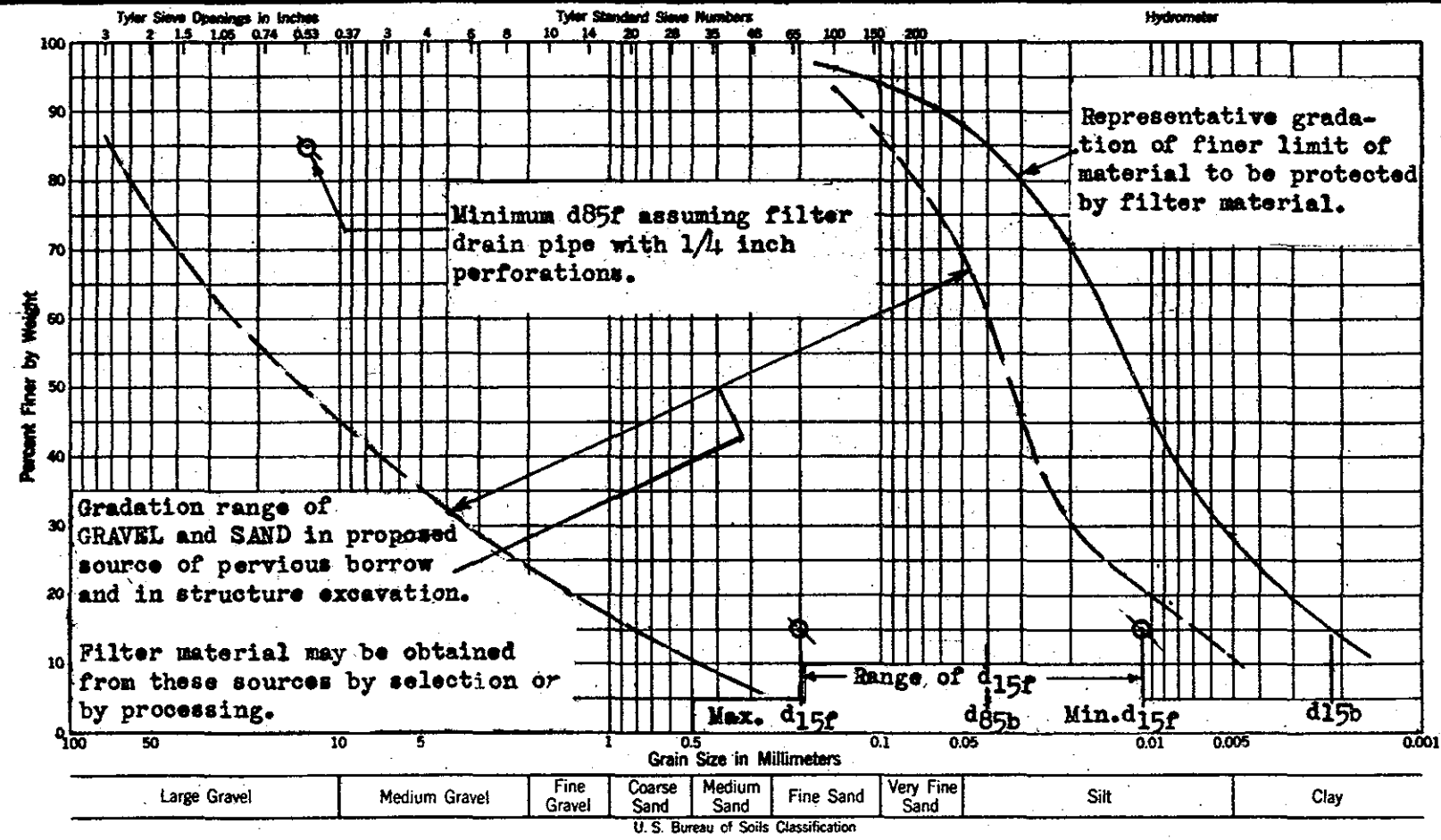
APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION
APPROVED: Julian L. Potter, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER
SUBMITTED: A. T. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER
DES. BY W. L. Shannon, ENGINEER; DR. BY; IN. BY; TR. BY; CH. BY

FILE NO. M19-13/35

PLATE II-13

WAR DEPARTMENT

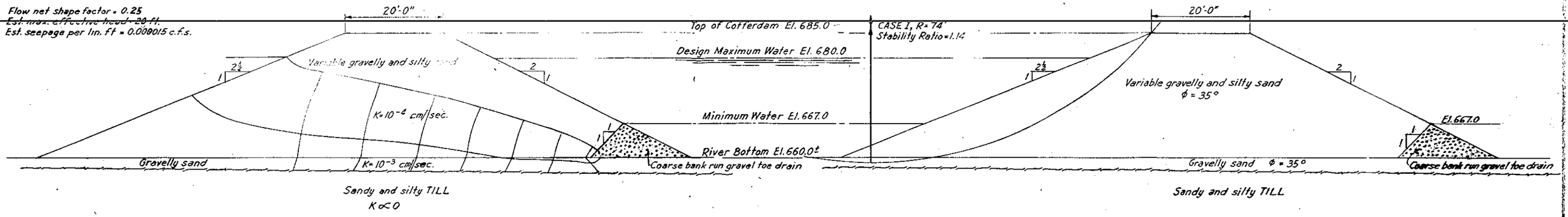
CORPS OF ENGINEERS U.S. ARMY



BENNINGTON DAM-FILTER ANALYSIS

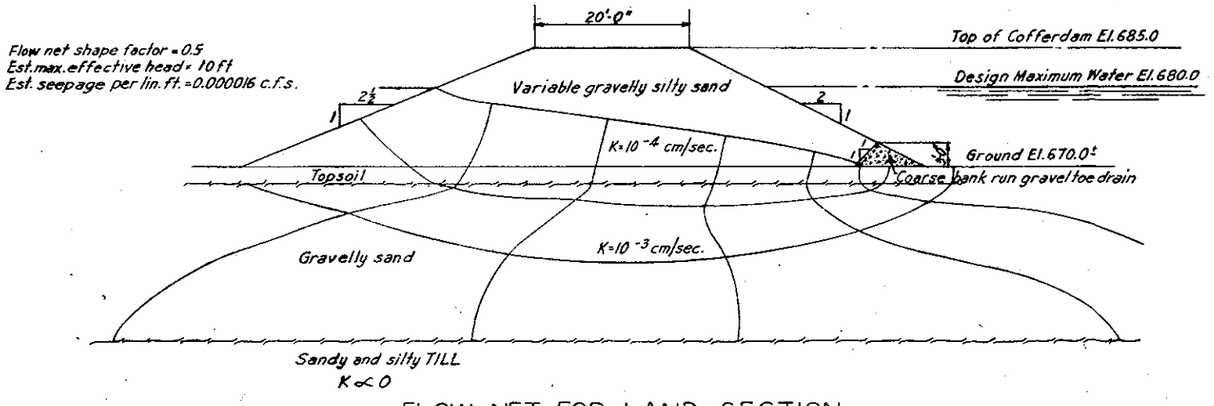
FILE NO M19-13/36

PLATE II-14



FLOW NET FOR RIVER SECTION

STABILITY ANALYSIS FOR RIVER SECTION



FLOW NET FOR LAND SECTION

Elevations are based on U.S.C. & G.S. datum Mean Sea Level.

THIS DRAWING REDUCED TO ONE HALF THE ORIGINAL SCALE

NO.	CHARACTER	DATE	APP'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
STABILITY AND SEEPAGE ANALYSIS
FOR UPSTREAM COFFERDAM

IN SHEETS SHEET NO. SCALE: 1 IN. = 10 FT.

U. S. ENGINEER OFFICE, BOSTON, MASS. APRIL 1938

APPROVAL RECOMMENDED: John Z. Allen
Chief Engineering Division

APPROVED: Julian L. Schley
Division Engineer

SUBMITTED: J. J. Brown
Chief Design Section, Eng. Division

W. L. Shannon
Asst. Engr. Design Section

FILE NO. M19-13/37

PLATE II-15

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX III

HYDRAULIC DESIGN

To accompany definite project report
Dated April 1945

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX III - HYDRAULIC DESIGN

C O N T E N T S

Paragraph	Title	Page
a.	Outlets	III-1
b.	Spillway	III-3
c.	Stilling Basin	III-3
d.	Tailwater	III-4

PLATES

Plate	Title
III-1	Discharge Rating Curve of Single Conduit
III-2	Conduits - Hydraulic and Energy Gradients
III-3	Conduit Velocities
III-4	Tailwater Conditions During Conduit Discharges
III-5	Spillway Rating Curve
III-6	Design of Spillway Crests
III-7	Tailwater Rating Curves
III-8	Spillway Stilling Basin

APPENDIX III - HYDRAULIC DESIGN

a. Outlets.- (1) Purpose.- The outlets will be used initially to control freshet and flood discharges and the daily releases from the existing Powder Mill Reservoir of pondage required by the power dams in Bennington. If the dam is raised to its ultimate height, the outlets will be used also to control the releases of the additional conservation storage created to regulate the low-water stream flows for the benefit of the downstream developments and sanitation.

(2) Size and Invert Elevation.- The spillway crest of the existing downstream dam (Monadnock Power Dam) is at elevation 663.5, and with 2-foot flashboards creates a normal tailwater elevation of 665.5 at the proposed Bennington Dam. The crest of the upstream dam (Powder Mill Dam) is at elevation 675.0, but with existing flashboards a conservation pool at elevation 678.15 is obtained. To meet the requirements of these existing developments, the inverts of the outlets are located at elevation 667.0 which provides sufficient depth for the drawdown of the present conservation storage. This elevation places the inlets below the elevation of the normal reservoir surface, and the outlet portals above normal tailwater, thus reducing difficulties resulting from ice conditions which are likely to occur in this latitude. This elevation also creates a small drop from the outlet portals to the tailwater, which is advantageous as an aid in spreading the jet before it submerges into the stilling basin.

The size and number of outlets have been selected to satisfy the following conditions:

(a) To discharge normal daily flows.

(b) To discharge approximately 4000 c.f.s. with reservoir stage at initial and ultimate spillway crests without partial gate openings, for the control of flows during flood periods and for emptying the reservoir following floods.

(c) To provide flexibility for possible changes in operating requirements.

(d) To distribute the discharge over considerable width of the stilling basin to produce the hydraulic jump and dissipate the energy.

(e) To have additional conduits for emergency in case gates become inoperative, and to expedite the emptying period following a full reservoir.

To meet the above requirements, six gated conduits are provided, each warping from 4 feet wide and 6 feet high at the gate sections to 4'-6" wide and 5'-0" high at the outlet portal section. The cross-sectional area therefore decreases from 24 square feet through the mid-conduit section to 22.50 square feet at the portal as shown on Plate III-3. With this arrangement the hydraulic control will be at the portal and the possibility of cavitation will be a minimum. The reduction in height of portal roof and increase in width also produces an initial spread of the discharge jet that will continue into the stilling basin and produce a smaller initial depth (d_1). The invert of the conduit discharge curve becomes tangent to the toe of the spillway and is designed to be slightly flatter than the theoretical curve of the lower nappe of the discharge jet when operating under maximum head. Each conduit has the following discharge capacities for selected reservoir elevations:

Elevation		Discharge Capacity, c.f.s.
678	Normal Surface of Powder Mill Reservoir	400
705	Initial Spillway Crest	920
712	Ultimate Spillway Crest	1010

In the operation of the initial development, four outlets will normally be used for regulating flood flows and will discharge a maximum of 3680 c.f.s., and in the ultimate development, four outlets will be used with a maximum discharge of 4040 c.f.s. The gates will be operated to empty the reservoir as discussed in Paragraph x, of Appendix I, and as illustrated on Plate I-21.

(3) Discharge Capacity.- The discharge capacity of the outlets is based on computations summarizing the various head losses from trash rack, entrance, gate slots, friction and velocity head. An "n" value of 0.013 is assumed for the friction coefficient. The discharge capacity for each outlet is shown on Plate III-1. The hydraulic and energy gradients are illustrated on Plate III-2, with the average conduit velocities shown on Plate III-3. The discharge conditions from the conduits cannot be readily analyzed. The hydraulic jump curves shown on Plate III-4 are based on the assumption that the discharge jet, having the initial depth of d_1, will spread with a flare of 1 on 4. This flare is based on the accepted maximum flare of stilling basin walls both in model studies and prototypes. It is assumed further that the discharge will follow the concrete toe of the spillway and spread with a decreasing depth instead of "boring" into the tailwater. These assumptions are substantiated by comparable hydraulic model studies, particularly the "Model Study of the Spillway and Stilling Basin for the John Martin Dam, Arkansas River", Technical Memorandum No. 166-1, and the "Model Studies of the Spillway and Integral Sluices for the Canton Dam, North Canadian River", Technical Memorandum No. 190-1. The computations indicate that

under most unfavorable conditions the hydraulic jump will take place before the discharge spread from one conduit overlaps the discharge from the adjacent conduit. However, for optimum stilling basin conditions, the manual for the gate operation will stipulate that alternate gates will be opened until a fourth gate is required.

b. Spillway.- The spillway for the initial development with crest at elevation 705 is designed for a discharge of 45,900 c.f.s. with a length of 300 feet and an assumed "C" coefficient of 3.8, the maximum head on the weir is 11.8 feet. (Spillway Rating Curve, Plate III-5). The larger surcharge storage in the ultimate development results in a peak discharge of 42,200 with a head of 11.2 feet. The shape of the initial spillway crest conforms to the exponential curve recommended in Circular Letter No. 3281, dated September 1944, relative to that subject. This exponential function is expressed as follows:

$$X^{1.85} = 2H_c^{0.85} Y$$

where X = horizontal distance from ogee crest line
Y = vertical distance below ogee crest level
H_c = design head on ogee crest (does not include velocity of approach head).

The curve from the upstream face of the dam to the crest is a compound curve conforming to similar recommendations in the above Circular Letter. Plate III-6 shows the proposed spillway crests for the initial and ultimate developments. As it is not possible mathematically to have both initial and ultimate crests conform exactly to this function and have both tangent to the downstream slope of the spillway, the initial crest is designed to conform to the recommended exponential function, while the ultimate deviates slightly. The proposed ultimate shape is plotted on Plate III-6 with the theoretical function for comparison. The difference is so small that no difficulty from negative pressures or reduced "C" values is anticipated.

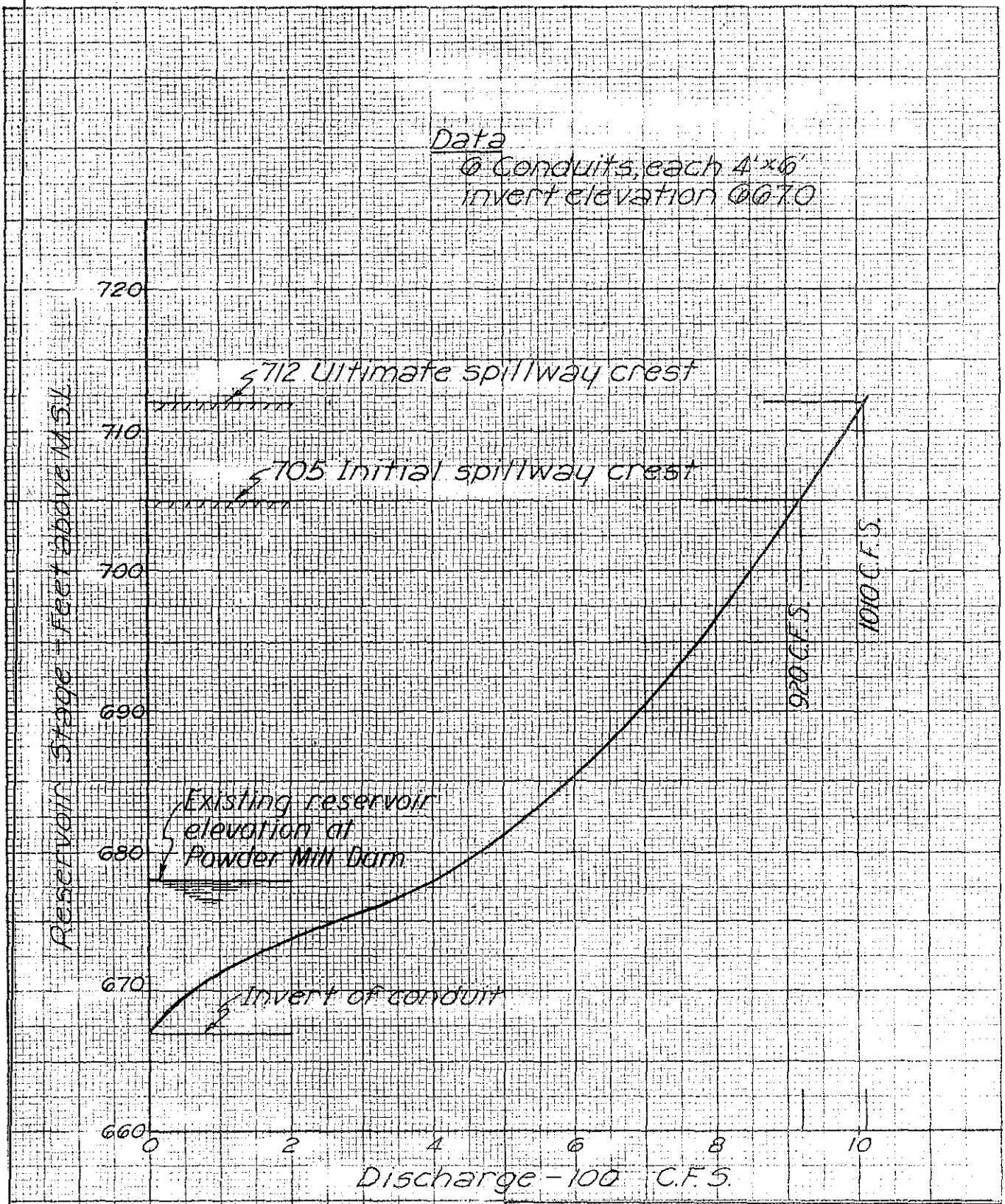
c. Stilling Basin.- The stilling basin is designed to produce a hydraulic jump to dissipate the energy of the maximum spillway discharge occurring during the most unfavorable tailwater conditions. Contrary to first conception, this design criteria is provided by the discharge during the initial development (crest elev. 705) instead of the ultimate development (crest elev. 712) due to greater discharge per foot of length in the initial development which more than offsets the effect of the added head obtained in the ultimate stage. The required depth of tailwater is derived by the formula for the hydraulic jump in rectangular channels:

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{d_1^2}{4} + \frac{2d_1 v_1^2}{g}}$$

where d_1 = depth before jump
v_1 = velocity before jump
d_2 = depth after jump
g = acceleration of gravity

The computed depth (d_2) required to satisfy this formula is approximately 23 feet. However, as model studies indicate quite conclusively that stilling basins constructed with baffles and end sills function satisfactorily with less than the theoretical required tailwater depth, the stilling basin floor is established by providing approximately 90 per cent of the computed depth. (See Plate III-8). The length of the stilling basin is approximately 4 times the tailwater depth, which is a conservative relationship to minimize the erosion of material in the discharge channel. The average velocity of the flow discharging from the stilling basin is approximately 8 feet per second. The size and spacing of the baffles follow the usual conventional pattern.

d. Tailwater.- The normal tailwater at the Bennington dam site is controlled by the Monadnock Power Dam about one half mile downstream. The crest of the overflow section of this dam is elevation 663.5 with flashboards at elevation 665.5. The abutment walls are constructed to elevation 673.2. The length of the spillway is 172 feet. Some water is used for generating power at the site but as this amount is small it has been neglected in computing the tailwater rating curve. The total computed discharge capacity of the spillway without overtopping the abutments is approximately 18,000 c.f.s. which slightly exceeds the maximum flow of record. For discharges in the range of the spillway design flood with river flows up to 50,000 c.f.s., it is necessary to consider that the river overflows its banks and overtops the abutments of the Monadnock Power Dam. A cross-section thru the Monadnock Power Dam and the adjacent topography was used as the hydraulic control for that reach of the river. Assuming that critical flow existed at this section a discharge rating curve was developed, and backwater computations made to determine the stages at the site of the proposed dam for corresponding discharges. The results of these backwater computations are indicated on Plate III-7. A second series of computations were then made assuming that with river flows in the magnitude of the spillway design flood, the Monadnock Power Dam would fail. Complete failure of the dam is entirely problematical for the dam consists of concrete founded on ledge outcrop although the abutments appear to be earth. However, assuming complete destruction of the dam results only in lowering the tailwater at the Bennington dam site approximately two feet for the river channel itself is restricted in its discharge capacity. Based on these computations it is estimated that the maximum tailwater at the Bennington Dam will vary between elevation 676 and 678.



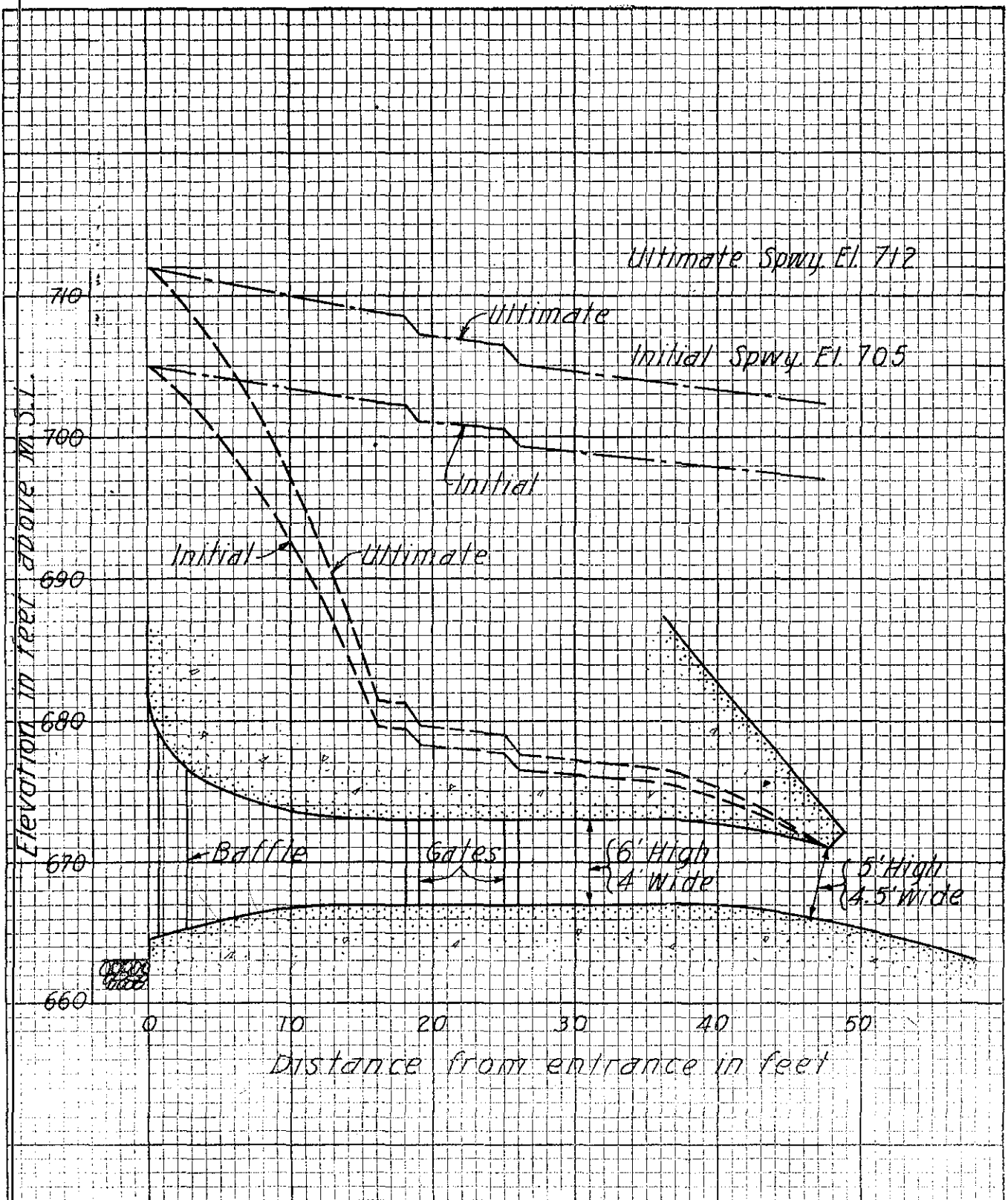
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

DISCHARGE RATING CURVE OF SINGLE CONDUIT

U.S. ENGINEER OFFICE
FILE NO. M 19-13/38

BOSTON, MASS.
18 APRIL 1945

PLATE III-1



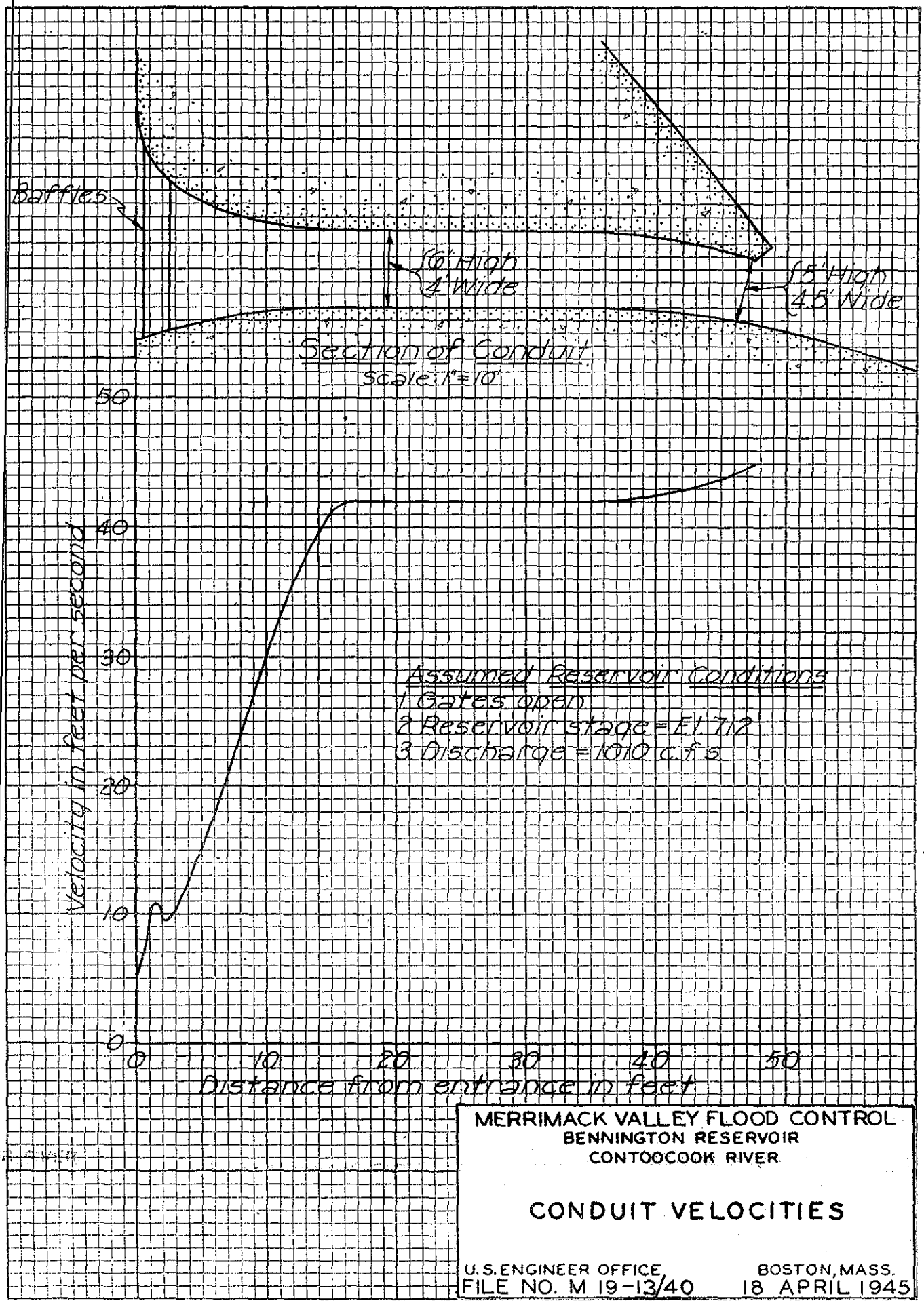
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

CONDUITS
HYDRAULIC & ENERGY GRADIENTS

U.S. ENGINEER OFFICE
FILE NO. M 19 - 13/39

BOSTON, MASS.
18 APRIL 1945

PLATE III-2



MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

CONDUIT VELOCITIES

U.S. ENGINEER OFFICE, BOSTON, MASS.
FILE NO. M 19-13/40 18 APRIL 1945

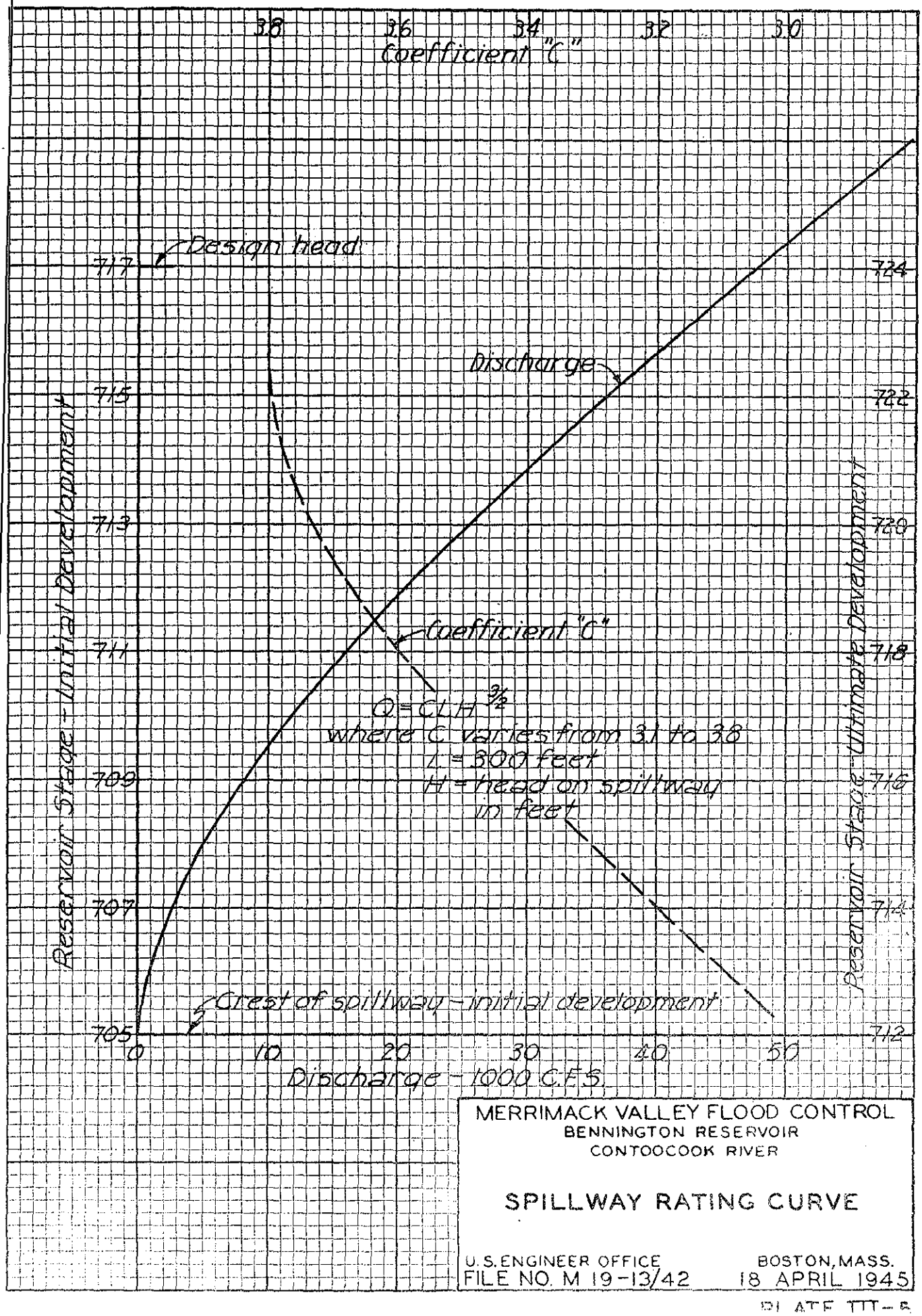
PLATE III-3

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

TAILWATER CONDITIONS DURING CONDUIT DISCHARGES

U.S. ENGINEER OFFICE
BOSTON, MASS.
FILE NO. M 19-12/41
18 APRIL 1945

PLATE III-4



MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

SPILLWAY RATING CURVE

U.S. ENGINEER OFFICE
FILE NO. M 19-13/42

BOSTON, MASS.
18 APRIL 1945

PLATE III-6

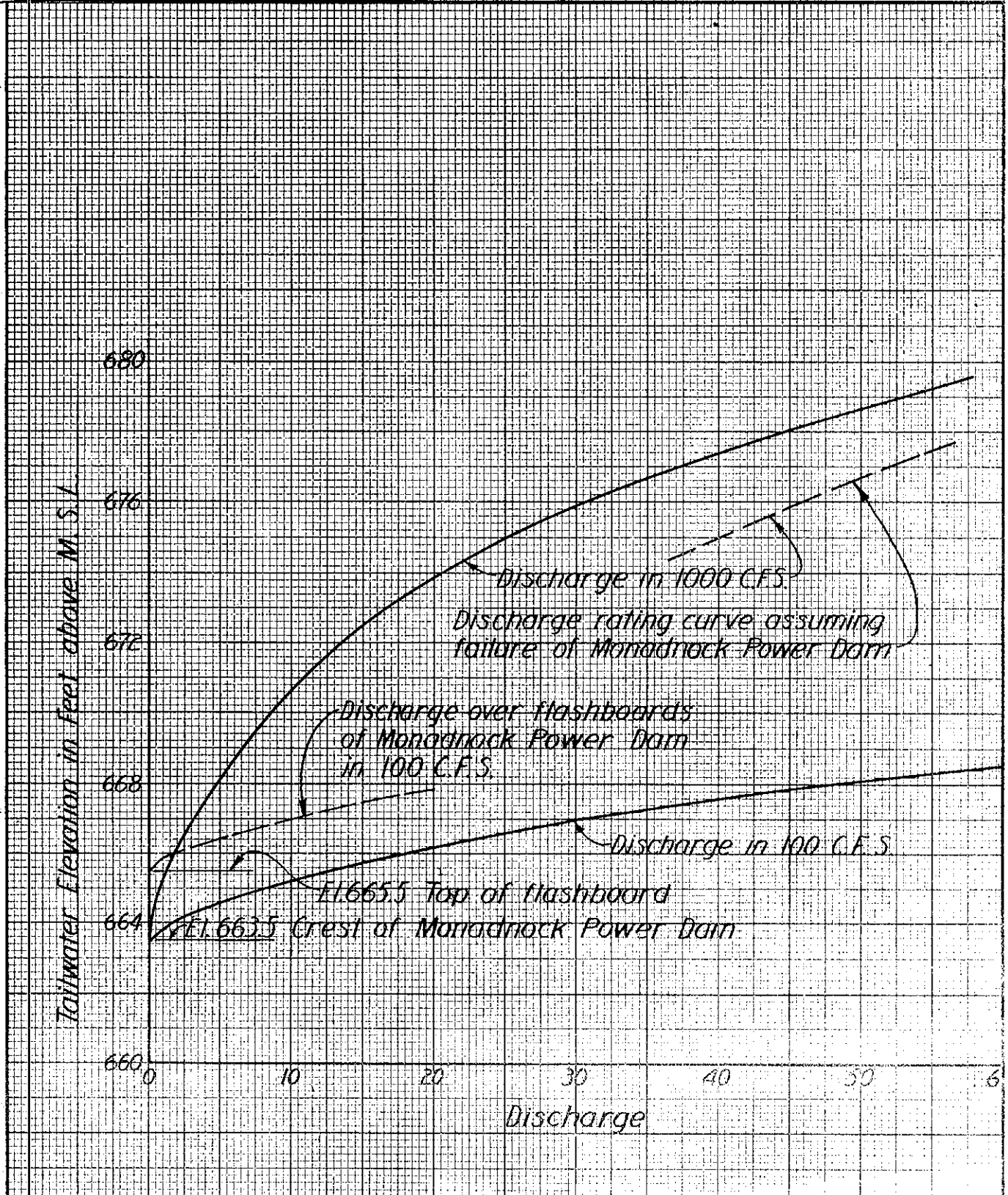
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

DESIGN OF SPILLWAY CRESTS

U.S. ENGINEER OFFICE
FILE NO. M 19-13/43

BOSTON, MASS.
18 APRIL 1945

PLATE III-6



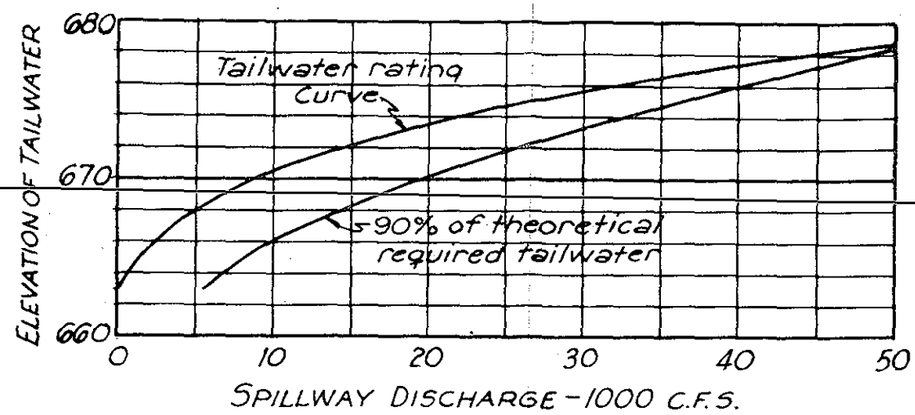
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

TAILWATER RATING CURVES

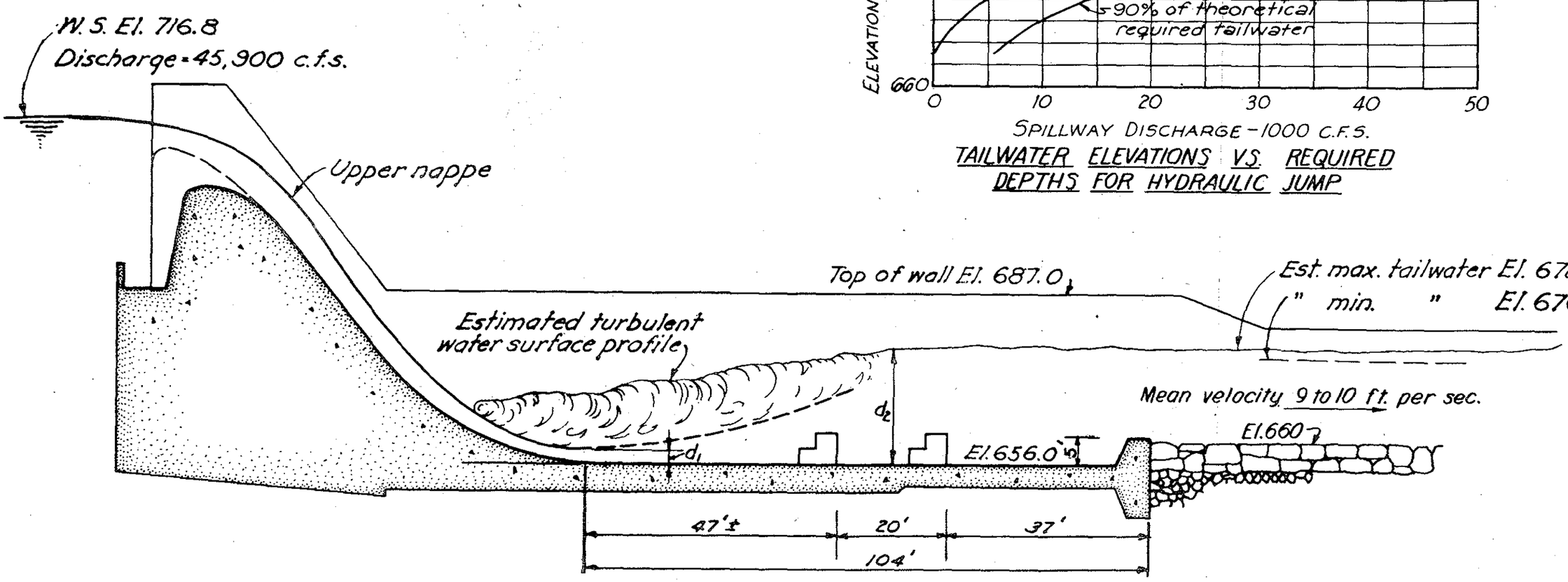
U.S. ENGINEER OFFICE
FILE NO M 19-13/44

BOSTON, MASS.
18 APRIL 1945

PLATE III-7



TAILWATER ELEVATIONS VS. REQUIRED DEPTHS FOR HYDRAULIC JUMP



PROFILE OF STILLING BASIN
Scale: 1" = 20'

Formula for hydraulic jump $d_2 = -\frac{d_1}{2} + \sqrt{\frac{2v_1^2 d_1}{g} + \frac{d_1^2}{4}}$

where d_1 = Depth above hydraulic jump in feet
d_2 = Depth below " " " "
g = Acceleration due to gravity.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

SPILLWAY STILLING BASIN

U.S. ENGINEER OFFICE
FILE NO. M 19-13/45

BOSTON, MASS.
18 APRIL 1945

PLATE III-6

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX IV

STRUCTURAL DESIGN

To accompany definite project report
Dated April 1945

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX IV - STRUCTURAL DESIGN

C O N T E N T S

Paragraph	Title	Page
a.	Selection of Structures	IV-1
b.	Consultants Conferences	IV-1
c.	Spillway	IV-2
d.	Non-overflow Sections	IV-4
e.	Stilling Basin	IV-4
f.	Stilling Basin Walls	IV-5
g.	Equipment House	IV-5
h.	Embankment	IV-6

PLATES

Plate	Title
IV-1	General Plan of Dam
IV-2	Profile and Sections of Dam
IV-3	Elevations and Sections
IV-4	Plans, Sections and Details
IV-5	Stability Analysis - Masonry Structures
IV-6	Schedule of Construction Operations

APPENDIX IV - STRUCTURAL DESIGN

a. Selection of Structures.- Because of the heterogeneous foundation conditions and depth to bedrock existing throughout the entire area of the proposed dam site, numerous locations and arrangement of structures have been analyzed within the reach extending from the existing Monadnock Power Dam to a section approximately 1200 feet upstream from the Powder Mill Dam. Considerable foundation exploration work has been accomplished within the above noted reach (see Plate II-1), and the general plan of the dam proposed in this report (see Plate IV-1), is considered the most feasible and economical layout with both the initial and ultimate developments in view. As shown on the geological profiles, Plates II-2 and II-3, the highest impervious glacial till deposit that extends in depth to the underlying bedrock occurs on the east bank of the river and, therefore, the spillway, stilling basin and masonry structures have been located within this area in order to obtain a suitable impervious foundation. The gate controlled outlets have been located in the spillway section which eliminates a separate outlet structure and presents a functional and compact arrangement of structures. Although this spillway location necessitates considerable excavation for an approach and discharge channel, all material, with the exception of stripping, can be utilized in the embankment section as noted in Appendix II. Gravity type non-overflow sections extending into the earth embankment on either side of the spillway have been used in lieu of high retaining walls at a substantial saving in cost. An earth embankment section of rolled earth fill extends to high ground on either side of the spillway, and the impervious core cutoff extends into the glacial till foundation with the exception of that portion of the embankment between Sta. 28 + 15 and 36 + 90 on the westerly side where the till tapers out. An inspection trench has been provided under the core and provisions made for the ultimate addition of an impervious blanket on the upstream side where the cutoff does not contact till between the above noted stations. A relief well system has been designed for a portion of the downstream side of the west embankment in order to relieve the hydrostatic head due to percolation through the silty sand. Ample borrow suitable for concrete aggregates and the pervious fill and filter requirements of the embankment is available within a half mile radius. Rock borrow can be obtained approximately three miles from the dam site.

b. Consultants Conferences.- During the progress of the design, two Board of Consultants meetings were held in the Boston District Office and the selection of structures and features of design were discussed in detail. The first conference was held on 1 September 1944 and the following members were present:

Mr. W. H. McAlpine - Office, Chief of Engineers, Washington, D.C.
Dr. Arthur Casagrande - Harvard University, Cambridge, Mass., Consultant
Mr. W. F. Uhl - Charles T. Main, Inc., Boston, Mass., Consultant

The second conference was held on 14 - 15 December 1944, and the following members were present:

Mr. W. H. McAlpine, Office, Chief of Engineers, Washington, D.C.
Mr. W. F. Uhl - Charles T. Main, Inc., Boston, Mass., Consultant
Mr. J. D. Justin - Philadelphia, Pennsylvania, Consultant
Dr. Arthur Casagrande - Harvard University, Cambridge, Mass., Consultant

Informal discussions were held at various times with Mr. W. H. McAlpine and Dr. Arthur Casagrande on various features of design.

Prior to the completion of the final contract plans, one more meeting of the Board of Consultants will be held in order to review the contract plans.

c. Spillway.- The spillway structure is a wide-base section designed to withstand the forces that will be applied to the ultimately constructed dam, and is the result of a study of several types of sections. First, comparative estimates were made of a concrete hollow type and a concrete gravity type of structure which indicated that the gravity type of section is less expensive. The gravity section has the further advantage that it would be less affected by the severe climatic conditions of this vicinity. Second, a solid concrete gravity section was proposed with an upstream cutoff to increase the path of percolation of the head water and decrease the uplift, and also provide resistance to sliding on the till foundation. This section did not meet the approval of the Board of Consultants as it was the consensus of opinion that due to the high ground water in the vicinity of the spillway, the construction of a cutoff would impose too many construction difficulties. Therefore, a nearly flat based concrete gravity spillway structure, sections of which are shown on Plate IV-4, was adopted as the most feasible design.

The stability analysis for the gated and ungated sections with the reservoir full is based on a water level in the reservoir equivalent to the extreme high water resulting from the spillway design flood and corresponding maximum tailwater (see Plate IV-5). Uplift pressures have been determined from flow net studies (see Plate II-13) and are equal to the full effective head applied to 100% of the base. Under these conditions, the resultant falls within the middle third of the base and gives a maximum base pressure, with the reservoir full, of 5420 pounds per square foot for the ungated section and 5310 pounds per square foot for the gated section. The stability analysis with the

reservoir empty, gives a maximum base pressure of 8600 pounds per square foot for the ungated section and 8300 pounds per square foot for the gated section, which values are judged to be well within the allowable load limit for the till foundation. The resultants for the gated and ungated sections have also been computed with earthquake-loadings combined with the normal pressures for the reservoir full and empty. In this case, the resultant computed for the reservoir empty falls within the middle third, and for the reservoir full, falls just outside the third point for both the gated and ungated sections. However, due to the remoteness of any possible earthquake shock to the dam in this location when the reservoir is full, the sections were not increased to make the resultant fall within the middle third of the base with earthquake loads applied.

In order to obtain a minimum factor of safety against sliding, it was necessary to increase the size of the spillway somewhat in order to obtain a greater vertical component of weight. The shearing strengths of the till used in computing the resistance to sliding were obtained from actual laboratory tests and are shown on Plate II-11. The spillway bears directly against the stilling basin slab and therefore the resistence to sliding of the first monolith of the stilling basin slab, submerged, has been added to that of the spillway in computing the spillway sliding resistance. The resulting factor of safety against sliding is 1.51 for both the gated and ungated sections.

The outlet conduits have been provided with cast iron liners in the vicinity of the gates to prevent cavitation of the concrete. Air vents with intakes in the non-overflow section will relieve possible negative pressures in the immediate vicinity of the gate slots.

At the recommendation of the Board of Consultants, a wide cut with a trench for water collection will be provided on the upstream side of the spillway during construction. The base of the spillway has been sloped to allow the free drainage of any seepage water in the foundation prior to the placing of the base course of concrete. It is also proposed to place a mat of 12 inches of concrete over the entire foundation area of the spillway as soon as possible after excavation to prevent the foundation material from becoming soft and to facilitate the movement of equipment during the placing of the remainder of the concrete.

A removable section of concrete, as indicated in detail on Plate IV-4, has been provided which will permit the ultimate addition to the spillway to be added with a minimum amount of chipping.

The spillway has been divided into 10 monoliths of 30 feet each and, in accordance with the recommendations contained in the Engineering Manual for Civil Works, no keys have been provided in the construction, expansion, or contraction joints of the masonry structures.

d. Non-overflow Sections.- It was originally proposed to provide full height masonry walls of a counterfort type at the ends of the spillway weir to retain the embankment fill. Further study and analysis indicated that a saving of approximately $68,000 could be obtained by using a concrete gravity non-overflow section, extending from the spillway into the earth embankment, thus eliminating the high retaining walls and considerable fill. Plate IV-3 illustrates the adopted design.

The non-overflow section is designed on the basis of the proposed ultimate construction and provides for the addition of six feet to the top section. The section chosen for analysis is in the monolith adjacent to the spillway (see Plate IV-5). The stability analysis as shown on Plate IV-5 is based on the maximum water level reached during a spillway design flood and with maximum tailwater. Uplift pressures have been determined from the flow net (see Plate II-13), and are equal to the full effective head applied to 100% of the base. With these loadings applied, the resultant falls within the middle third of the base and gives a maximum foundation pressure of 8400 pounds per square foot. The computed resultant for the section, with the reservoir empty, falls within the middle third and gives a maximum base pressure of 8700 pounds per square foot which is within the allowable limit. The factor of safety against sliding is 1.52. With the additional earthquake loads applied, the resultant falls within the middle third of the base for the reservoir empty and outside the middle third for the reservoir full. As in the case of the spillway, the section has not been increased to bring the resultant back within the middle third with the reservoir full.

The construction procedure for the non-overflow section will be the same as for the spillway. The base of the non-overflow section has been sloped to allow the free drainage of any seepage water in the foundation prior to the placing of the one-foot base course of concrete, and has been divided into monoliths varying between 33 feet and 38 feet in length. A wide cut with a trench for water collection will be provided on the upstream side and a one-foot mat of concrete will be poured for the base of the section as soon as the excavation has been completed.

e. Stilling Basin Slab.- The stilling basin slab is a reinforced concrete slab with a one-foot base course of porous concrete founded on a mat of screened gravel and sand for drainage and the reduction of uplift. The design of the slab is based on a slab thickness computed to withstand the uplift and impact pressures to which it is subjected with an additional increment added for cavatation and spalling.

In order to reduce the uplift pressures, the stilling basin is drained by a 6-inch layer of sand and 6-inch layer of screened gravel founded on till on which a one-foot layer of porous concrete is poured as a base for the structural slab. Three perforated tile pipes serving

as collectors are located transversely at the third points under the slab, and are connected with the sand and gravel layer. The drains then connect with the wells provided in the retaining walls which in turn discharge into the stilling basin thus providing a system for relieving any excess head. The slab has been divided into monoliths 30 feet wide and which vary from 35 feet to 45 feet in length. Conforming to the opinion of the Board of Consultants, no weep holes have been provided in the slab.

f. Stilling Basin Walls.- In view of the selection of gravity concrete non-overflow sections and the fact that estimates showed approximately equal cost for gravity and counterfort walls, a gravity concrete section affording greater durability was selected for the stilling basin walls.

A passageway leading from the equipment house to the spillway was incorporated into the design of the east wall for access to the gate operating chambers and adit on the west wall. One monolith of the east wall is also used as a foundation for the equipment house and is more fully discussed in Paragraph g.

The stability analysis for the wall section, as shown on Plate IV-5, is based on the lateral pressure of the earth retained to berm height and normal tail water occurring shortly after a design flood in which the saturation line would be 9-1/2 feet above normal tail water in the embankment fill. The uplift pressure used, is based on 100% of the tail water head applied at the toe and increasing uniformly to 100% of the hydrostatic head at the heel, based on the saturation line, applied to 100% of the base. Under these conditions, the resultant is held within the middle third of the base and the factor of safety against sliding is 2.26 based on the shearing strength of till obtained from the curve on Plate II-11.

In order to decrease the uplift and lateral pressure on the spillway walls immediately after drawdown, to allow free drainage of the embankment, and to stabilize the foundation immediately after excavation, it is proposed to pour one foot of porous concrete as a base course for the walls, except at the well sections. The bases of the walls have been sloped to allow for the drainage of any seepage water prior to the placement of the porous concrete. The maximum length of a wall monolith is 45 feet.

g. Equipment House.- The equipment house is composed of a superstructure above berm height and a basement structure located on the east spillway wall. The superstructure houses the standby generators, switch board, oil pumps, toilet facilities and a workroom equipped with a monorail and chain hoist for handling heavy equipment. This structure has a framework of structural steel with reinforced concrete roof and floor slabs. The side walls are of brick with face brick

exterior and are 12 inches thick as indicated in the plan and sections on Plate IV-4.

The basement of the structure is incorporated as part of one monolith of the stilling basin gravity wall with reinforced concrete walls and partitions, and houses the heating boiler, transformer, deep well pump, water pressure storage tank, and a storage room. The passageway to the gate operating chambers in the east stilling basin wall enters the basement of the equipment room.

The entire structure is supported on piers which extend downward to the back slope of the retaining wall and to individual pier footings which are integral with the wall as shown in the equipment house detail on Plate IV-4.

h. Embankment.- The design of the embankment is discussed in detail under Appendix II.

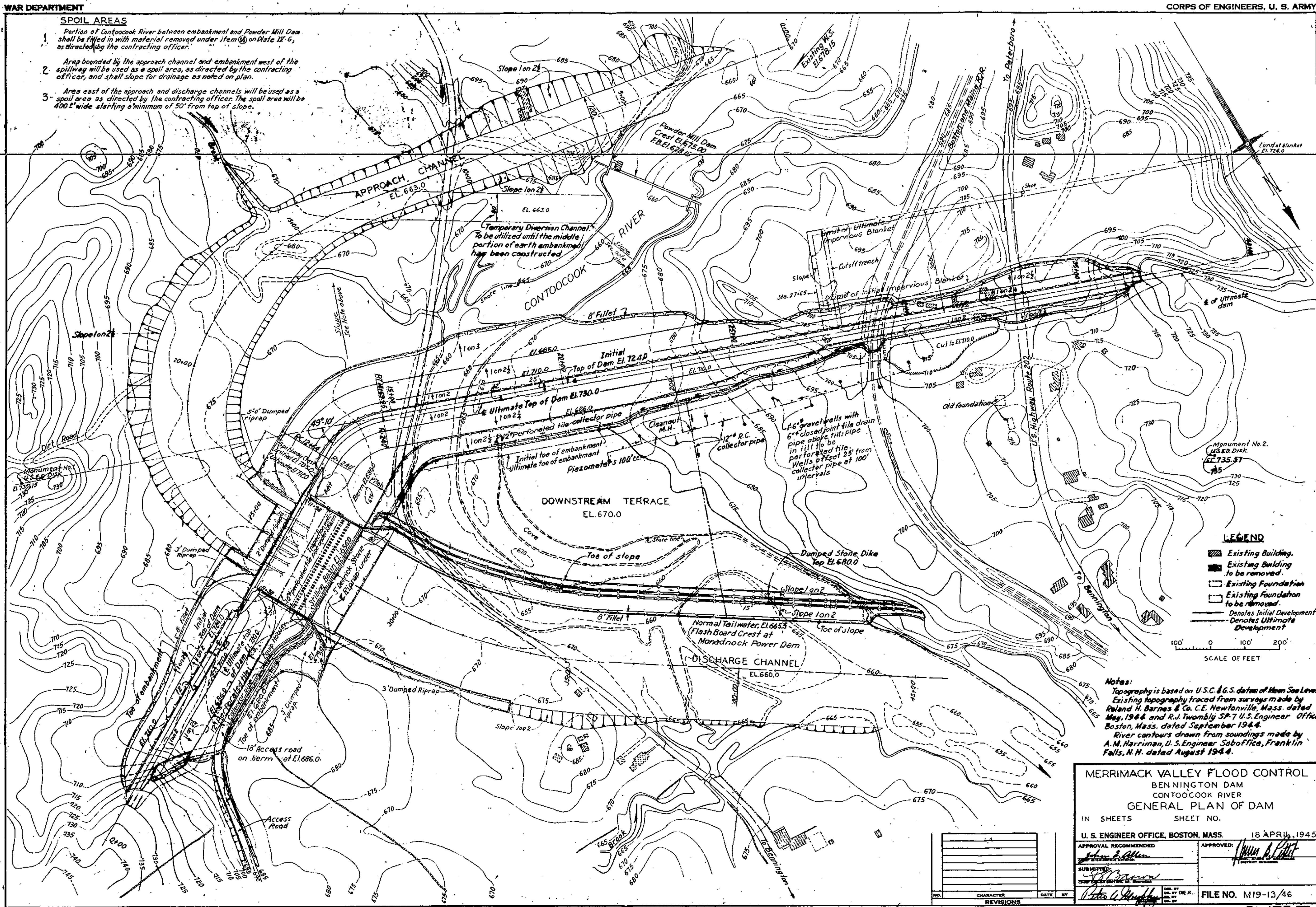
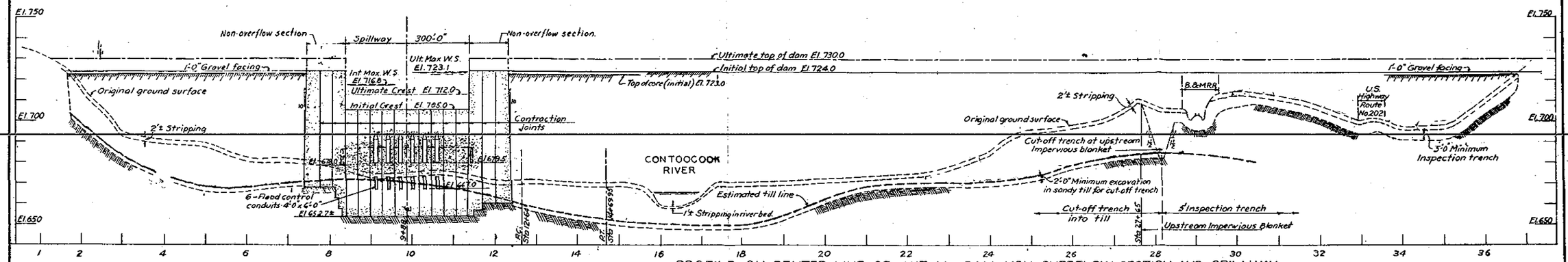

SPOIL AREAS
1 Portion of Contoocook River between embankment and Powder Mill Dam shall be filled in with material removed under item (14) on Plate IV-6, as directed by the contracting officer.
2. Area bounded by the approach channel and embankment west of the spillway will be used as a spoil area, as directed by the contracting officer, and shall slope for drainage as noted on plan.
3- Area east of the approach and discharge channels will be used as a spoil area as directed by the contracting officer. The spoil area will be 400± wide starting a minimum of 50' from top of slope.
APPROACH CHANNEL
EL. 663.0
Slope 1 on 2½
Powder Mill Dam Crest El. 675.00 F.B.E. 678.15
Existing W.S. El. 678.15
Temporary Diversion Channel. To be utilized until the middle portion of earth embankment has been constructed
CONTOOCOOK RIVER
Boston and Maine R.R.
To Peterboro
Limit of Ultimate Impervious Blanket
Limit of Initial Impervious Blanket
Cutoff trench
Initial Top of Dam El. 724.0
℄ Ultimate Top of Dam El. 730.0
8' Fillet
Perforated tile collector pipe
Cleanout M.H.
12" R.C. collector pipe
1'-6" gravel wells with 6" closed joint tile drain pipe above till; pipe in till to be perforated tile. Wells offset 25' from collector pipe at 100' intervals
Initial toe of embankment
Ultimate toe of embankment
Piezometers 100' c.c.
DOWNSTREAM TERRACE
EL. 670.0
Cove
Toe of slope
Dumped Stone Dike Top El. 680.0
Slope 1 on 2
Normal Tailwater El. 665.5 Flash Board Crest at Monadnock Power Dam
DISCHARGE CHANNEL
EL. 660.0
Dirt Road
Monument No. 1 U.S.E.D. Disk El. 735.15
Monument No. 2 U.S.E.D. Disk El. 735.57
Old foundation
U.S. Highway Route 202
To Bennington
℄ of Ultimate dam
Limit of blanket El. 726.0
5'-0" Dumped riprap
3' Dumped Riprap
Toe of embankment
18' Access road on berm at El. 686.0
Access Road
Brook
LEGEND
Existing Building.
Existing Building to be removed.
Existing Foundation
Existing Foundation to be removed.
Denotes Initial Development
Denotes Ultimate Development
100' 0 100' 200'
SCALE OF FEET
Notes:
Topography is based on U.S.C.&G.S. datum of Mean Sea Level.
Existing topography traced from surveys made by Roland H. Barnes & Co. C.E. Newtonville, Mass. dated May, 1944 and R.J. Twombly SP-7 U.S. Engineer Office, Boston, Mass. dated September 1944.
River contours drawn from soundings made by A.M. Harriman, U.S. Engineer Suboffice, Franklin Falls, N.H. dated August 1944.
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
GENERAL PLAN OF DAM
IN SHEETS SHEET NO.
U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL, 1945
APPROVAL RECOMMENDED
APPROVED:
SUBMITTED:
FILE NO. M19-13/46
NO. CHARACTER DATE BY
REVISIONS

PLATE IV-1

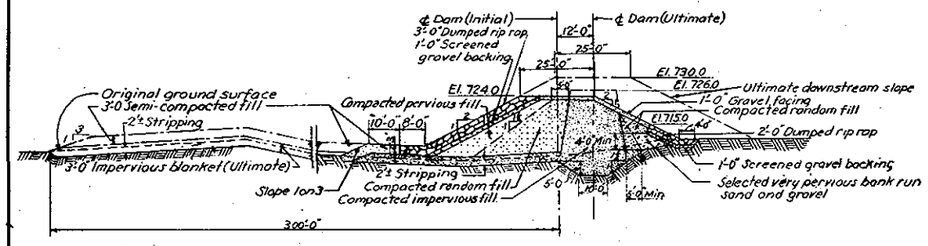
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

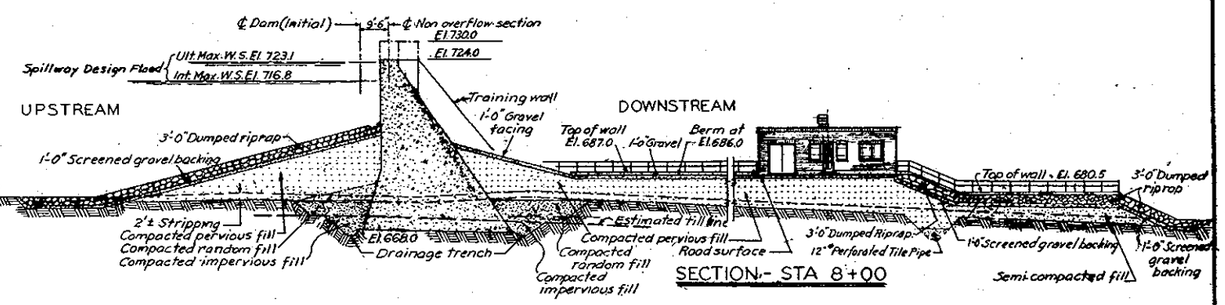


PROFILE ON CENTER LINE OF INITIAL DAM, NON OVERFLOW SECTION AND SPILLWAY

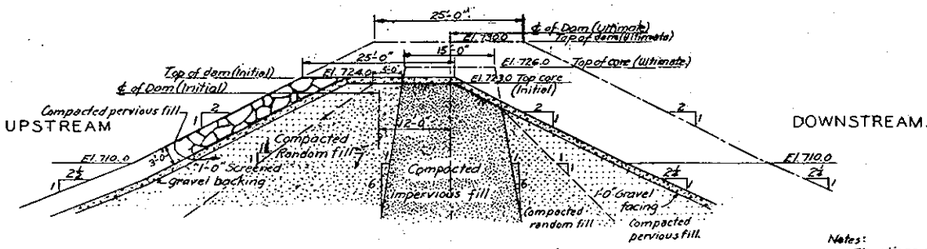
SCALE: HOR. IN HUNDREDS OF FEET
VERT. IN FEET



EMBANKMENT SECTION - STA. 32 +50

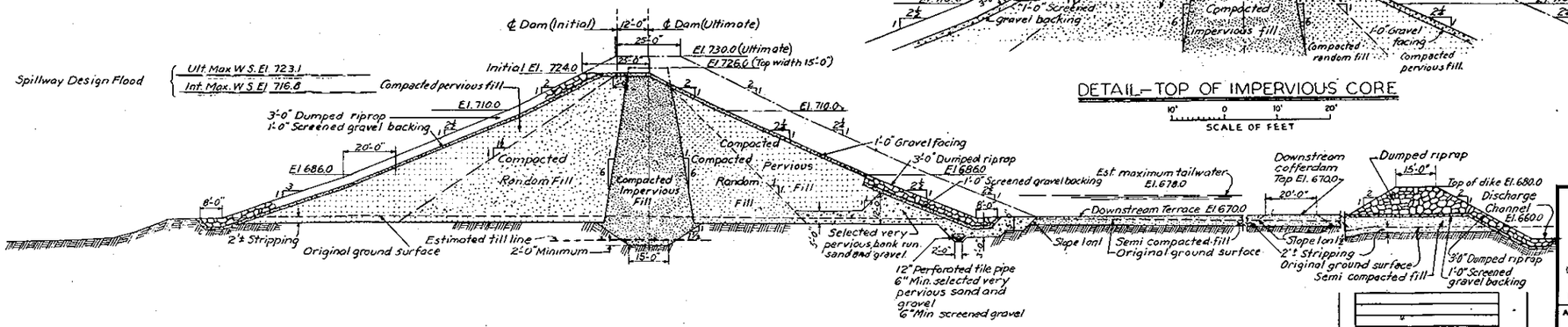


SECTION - STA 8+00



DETAIL - TOP OF IMPERVIOUS CORE

10' 0 10' 20'
SCALE OF FEET



EMBANKMENT SECTION - STA. 20 +00

Notes:
Elevations are based on U.S.C.& G.S. datum, Mean Sea Level
——— Denotes initial development
———·——·——· Denotes ultimate development

20' 0 20' 40'
SCALE OF FEET
(UNLESS OTHERWISE NOTED)

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
PROFILE AND SECTIONS OF DAM

IN SHEETS SHEET NO

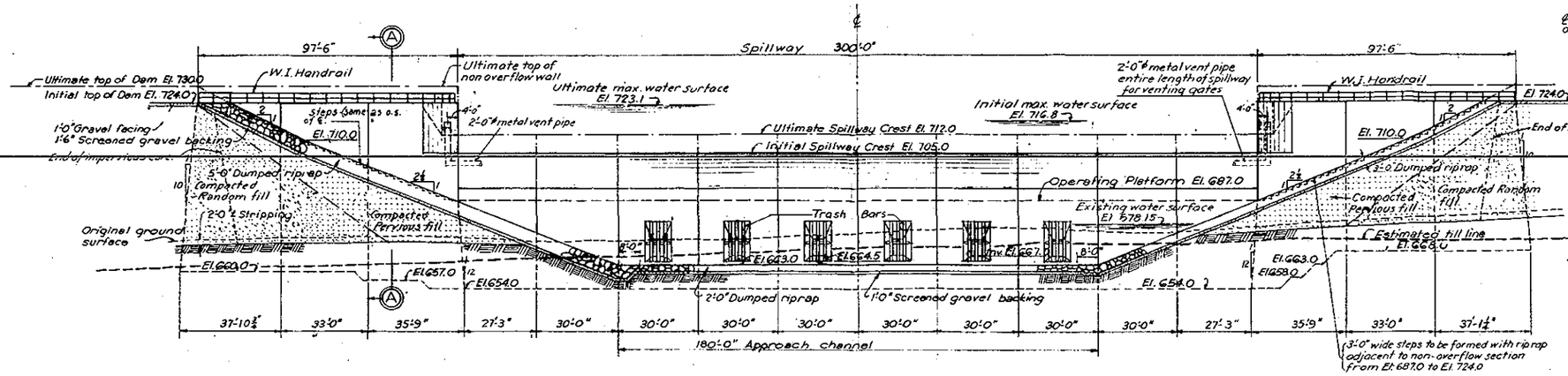
U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL, 1945

APPROVAL RECOMMENDED: Glen E. Allen, CHIEF ENGINEERING DIVISION
APPROVED: Jerry B. Post, DISTRICT ENGINEER
SUBMITTED: C. F. Brown, CHIEF DESIGN SECTION, DE. ENGINEER
Peter A. Mueller, DRN. BY, DE. BY, TR. BY, CH. BY

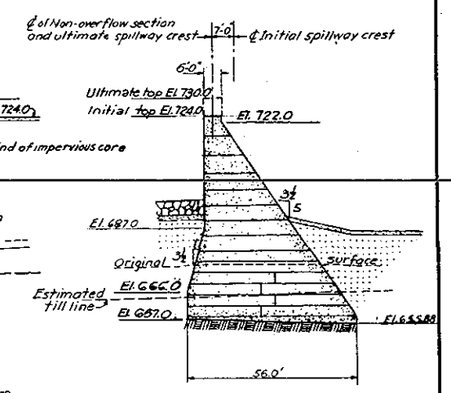
FILE NO. M 19-13/47

NO. | CHARACTER | DATE | BY
REVISIONS

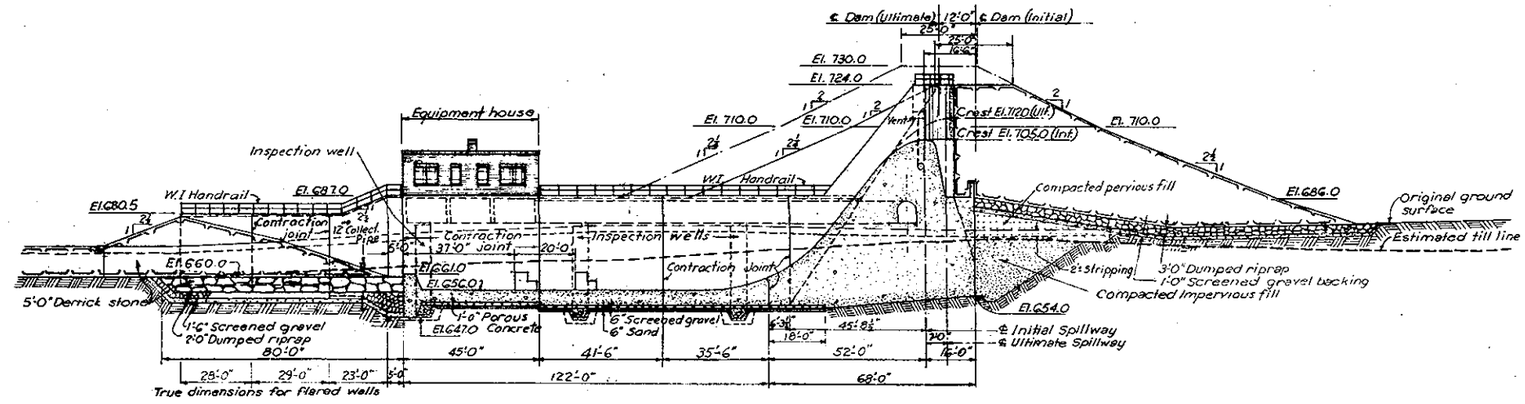
PLATE IV-2



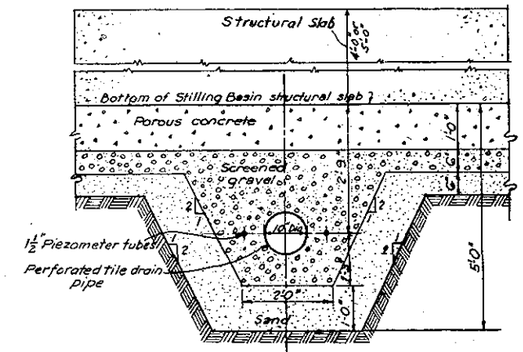
SECTION THRU APPROACH CHANNEL



SECTION A-A

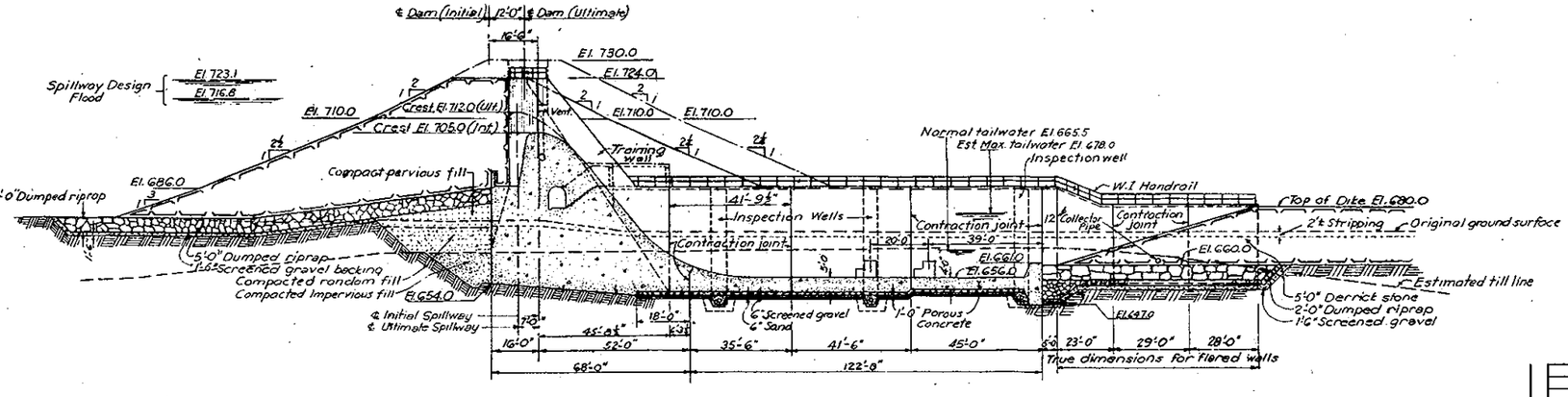


ELEVATION OF EAST WALL



TYPICAL DETAIL OF DRAIN UNDER STILLING BASIN

12" 0 1' 2' 3'
SCALE OF FEET



ELEVATION OF WEST WALL

Notes:
Elevations are based on U.S.C.&G.S. datum, Mean Sea Level.
———— Denotes initial development
———·———·——— Denotes ultimate development

20' 0 20' 40'
SCALE OF FEET
(UNLESS OTHERWISE NOTED)

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
ELEVATIONS AND SECTIONS
IN SHEETS SHEET NO

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED: John L. Gallen, Chief Engineering Division
APPROVED: Wilbur R. Smith, Colonel, Corps of Engineers, District Engineer
SUBMITTED: C. J. Brown, Chief Design Section, Sr. Engineer
Peter A. Muffly, Sen. Dr. Jr. Dr. Ch. Dr.

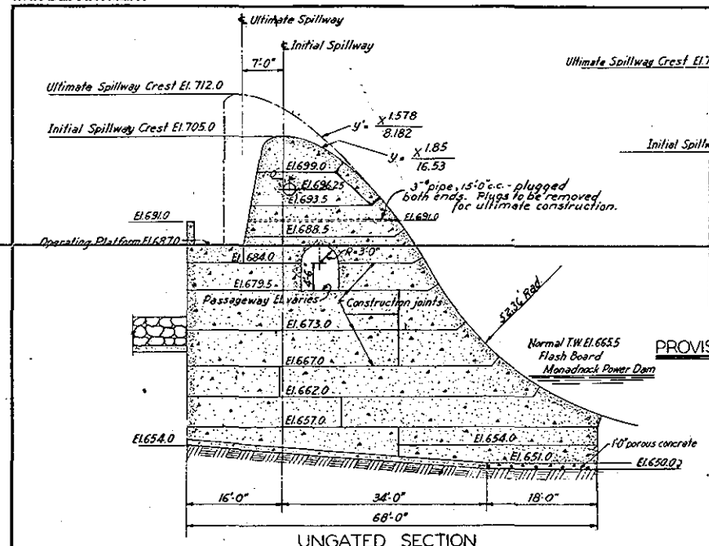
FILE NO. M19-13/48

NO.	CHARACTER	DATE	BY
REVISIONS			

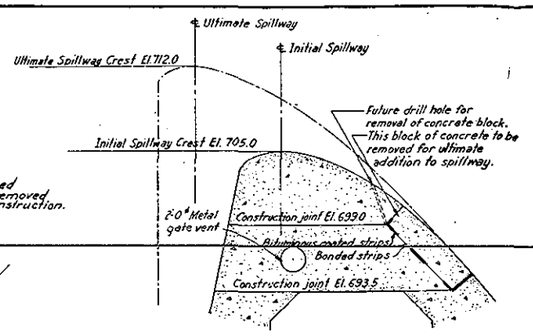
PLATE IV-3

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

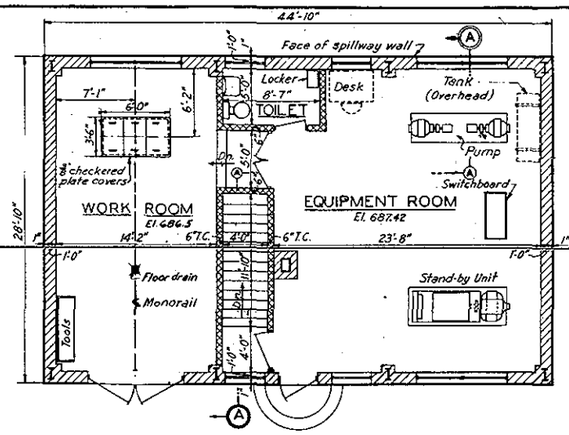


UNGATED SECTION

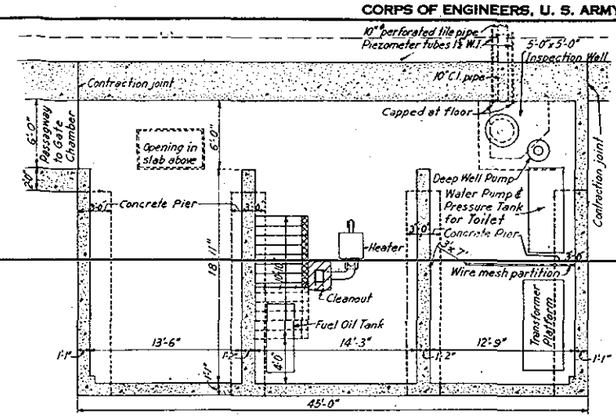


PROVISION FOR ULTIMATE ADDITION TO SPILLWAY

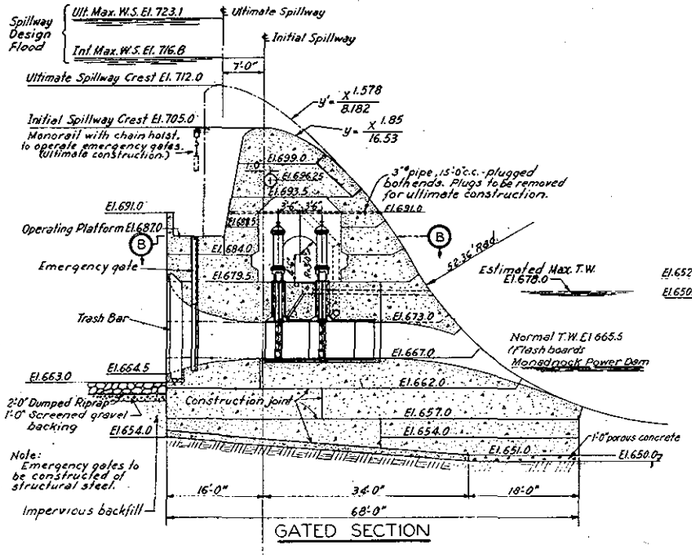
SCALE OF FEET



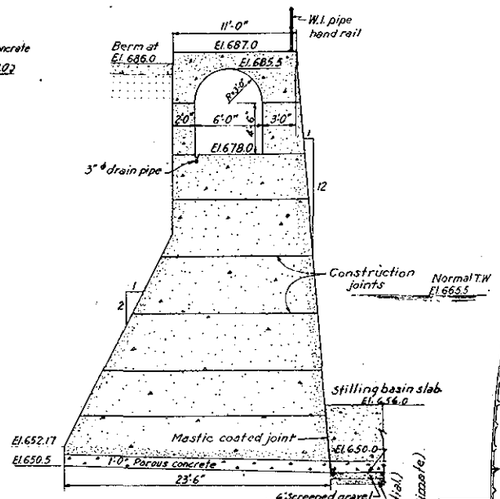
GROUND FLOOR PLAN



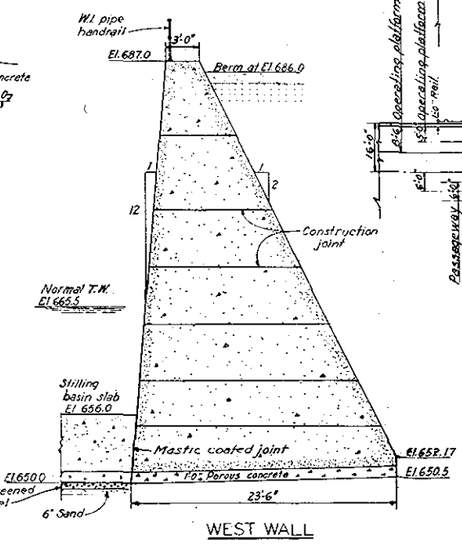
BASEMENT PLAN



GATED SECTION



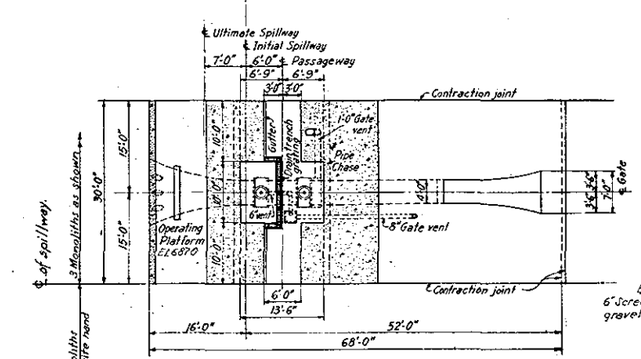
EAST WALL



WEST WALL

TYPICAL SPILLWAY WALL SECTIONS

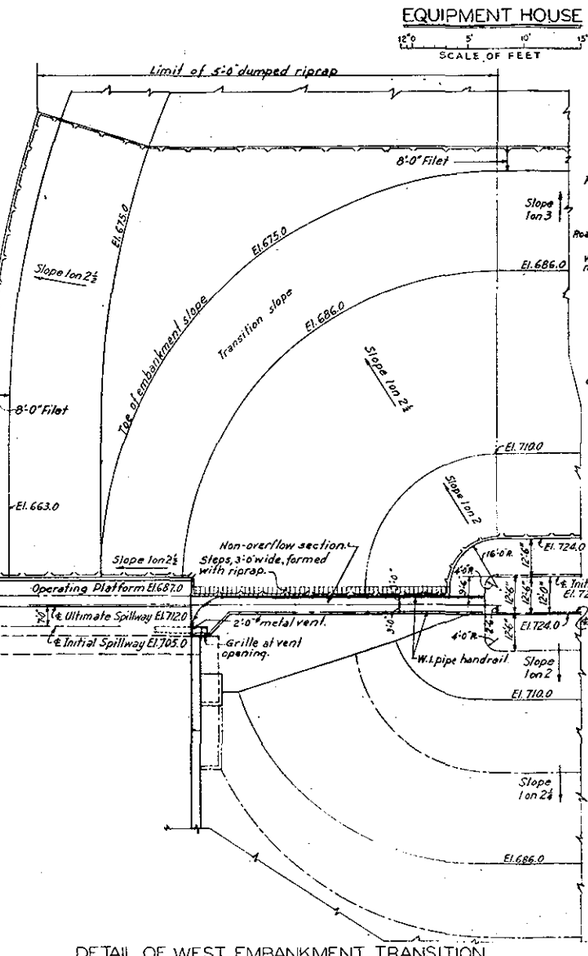
SCALE OF FEET



PLAN OF OUTLET WORKS AT B-B

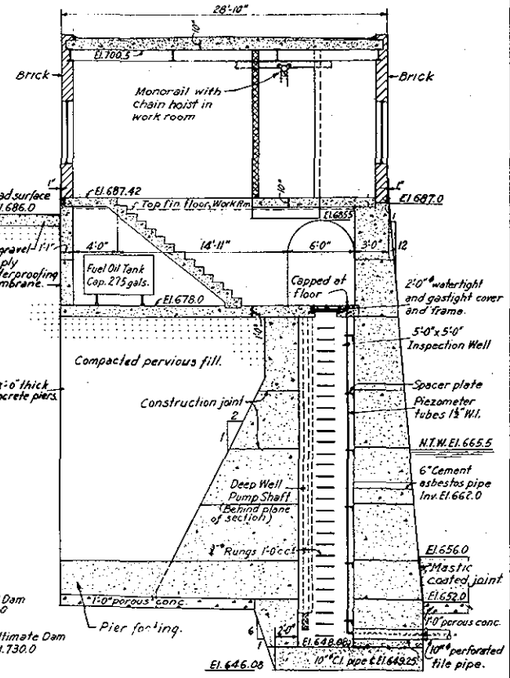
Note: Monolith shown is typical for 6 gated monoliths.

TYPICAL SPILLWAY SECTIONS



DETAIL OF WEST EMBANKMENT TRANSITION

SCALE OF FEET



SECTION A-A

SCALE OF FEET
(UNLESS OTHERWISE NOTED)

Note: Elevations are based on U.S.C. & G.S. datum of Mean Sea Level.

——— Denotes Initial Development.
———·——— Denotes Ultimate Development.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
PLANS, SECTIONS AND DETAILS

IN SHEETS SHEET NO.

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED
Chief Engineering Division

APPROVED:
Colonel, Corps of Engineers
District Engineer

SUBMITTED:
Chief Design Section, Eng. Division

No.	REVISIONS Character	Date	By

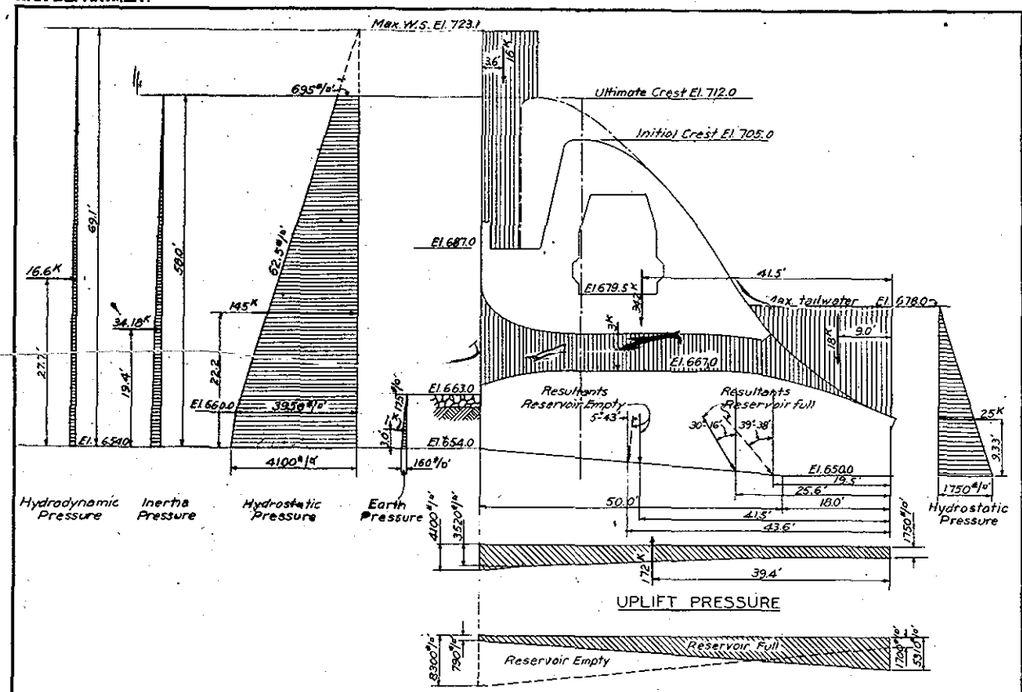
DES. BY
DR. BY D.H.R.
TR. BY
CH. BY

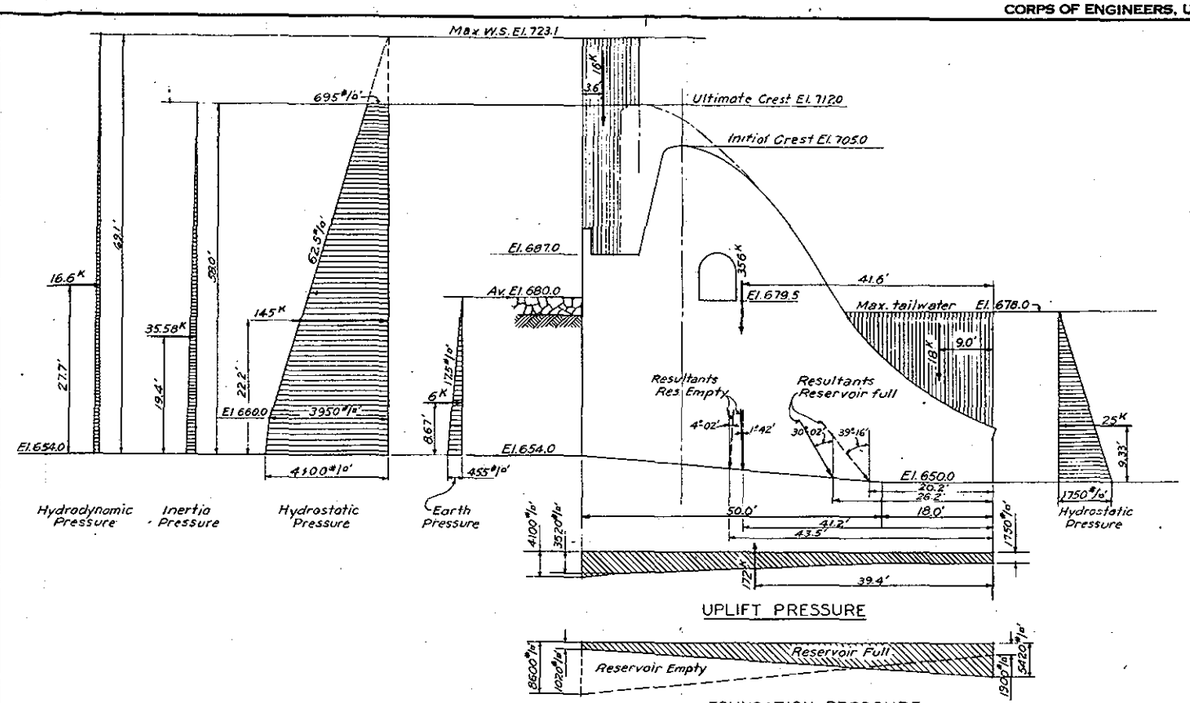
FILE NO. M19-13/49

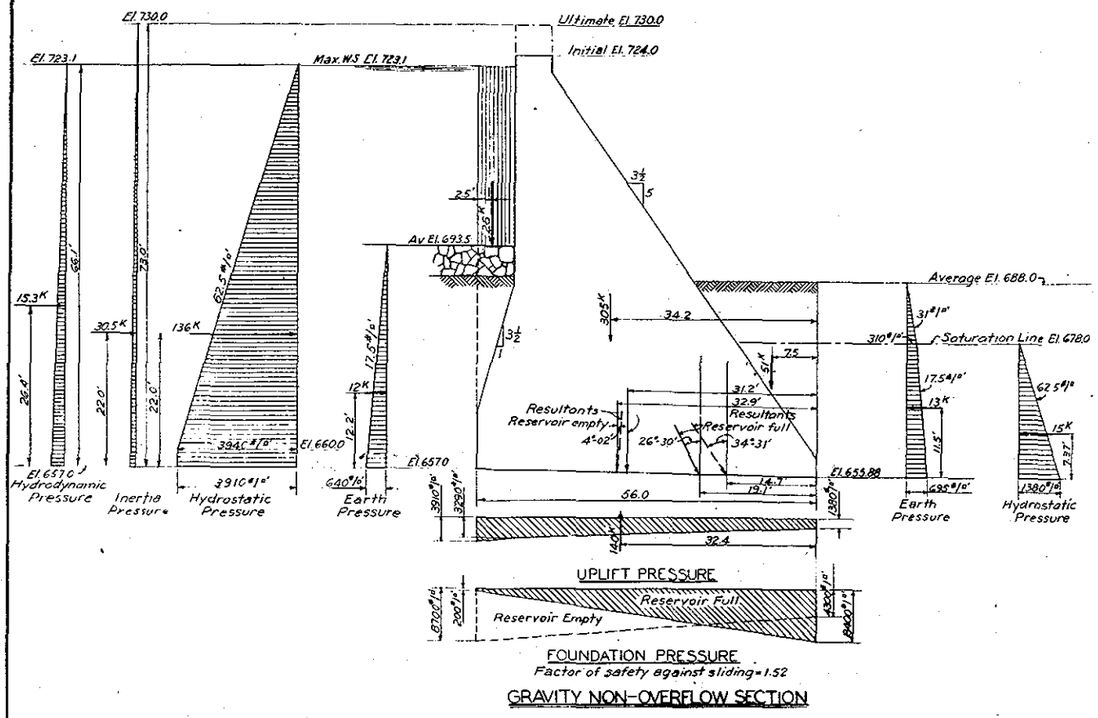
PLATE IV-4

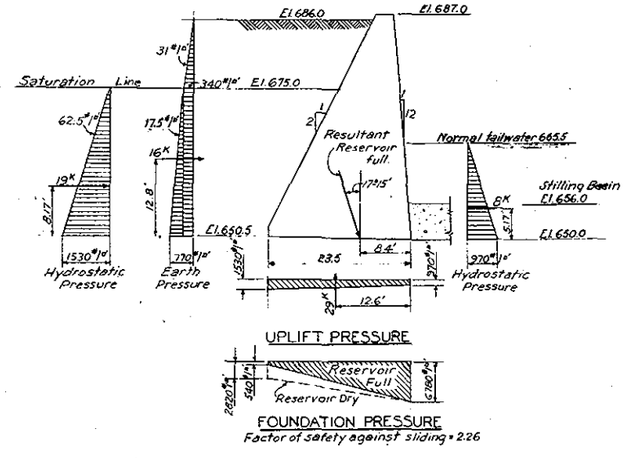
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY









DESIGN DATA

Scale of pressures:
Horizontal pressures; 1" = 2,000 pounds per sq. ft.
Foundation and uplift pressures; 1" = 10,000 pounds per sq. ft.

Design based on ultimate development. Top of Dam at El. 730.0 and Spillway Crest at El. 712.0

Reservoir pool assumed at Spillway design flood El. 723.1

Weight of concrete = 150 pounds per cu. ft.
Weight of water = 62.5 pounds per cu. ft.
Weight of shell material (dry) = 125 pounds per cu. ft.
Weight of shell material (submerged) = 80 pounds per cu. ft.
Weight of core material (dry) = 130 pounds per cu. ft.
Weight of core material (submerged) = 85 pounds per cu. ft.
Horizontal pressure of shell material (dry) = 31 pounds per sq. ft. per ft. of depth.
Horizontal pressure of shell material (saturated) = 80 pounds per sq. ft. per ft. of depth
Horizontal pressure of core material (dry) = 48 pounds per sq. ft. per ft. of depth.
Horizontal pressure of core material (saturated) = 90 pounds per sq. ft. per ft. of depth.

Pressures and loads indicated are average pressures and loads per foot of sections and are proportioned to the length of monolith on which they act

Spillway Uplift

Uplift pressures have been determined from the flow net study and are equal to the full effective head applied to 100% of the base area.

Spillway Sliding

The shearing strengths of till used in computing resistance to sliding were obtained from the curve on Plate II-11

The resistance to sliding of the first monolith of the Stilling Basin slab, submerged, has been added to the sliding resistance of the spillway in computing the spillway sliding factor of safety.

Foundation pressures and sliding factors do not include forces due to earthquake

Earthquake forces have been applied acting downstream for Full Reservoir condition and upstream for empty Reservoir condition

SCALE OF FEET 10' 0 10' 20'

LEGEND
Resultant without earthquake
Resultant with earthquake
Denotes Initial Development
Denotes Ultimate Development

REVISIONS

NO.	CHARACTER	DATE	BY

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM. CONTOOCOOK RIVER
STABILITY ANALYSIS
MASONRY STRUCTURES

IN SHEETS SHEET NO

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1945

APPROVAL RECOMMENDED
Chief Engineering Division

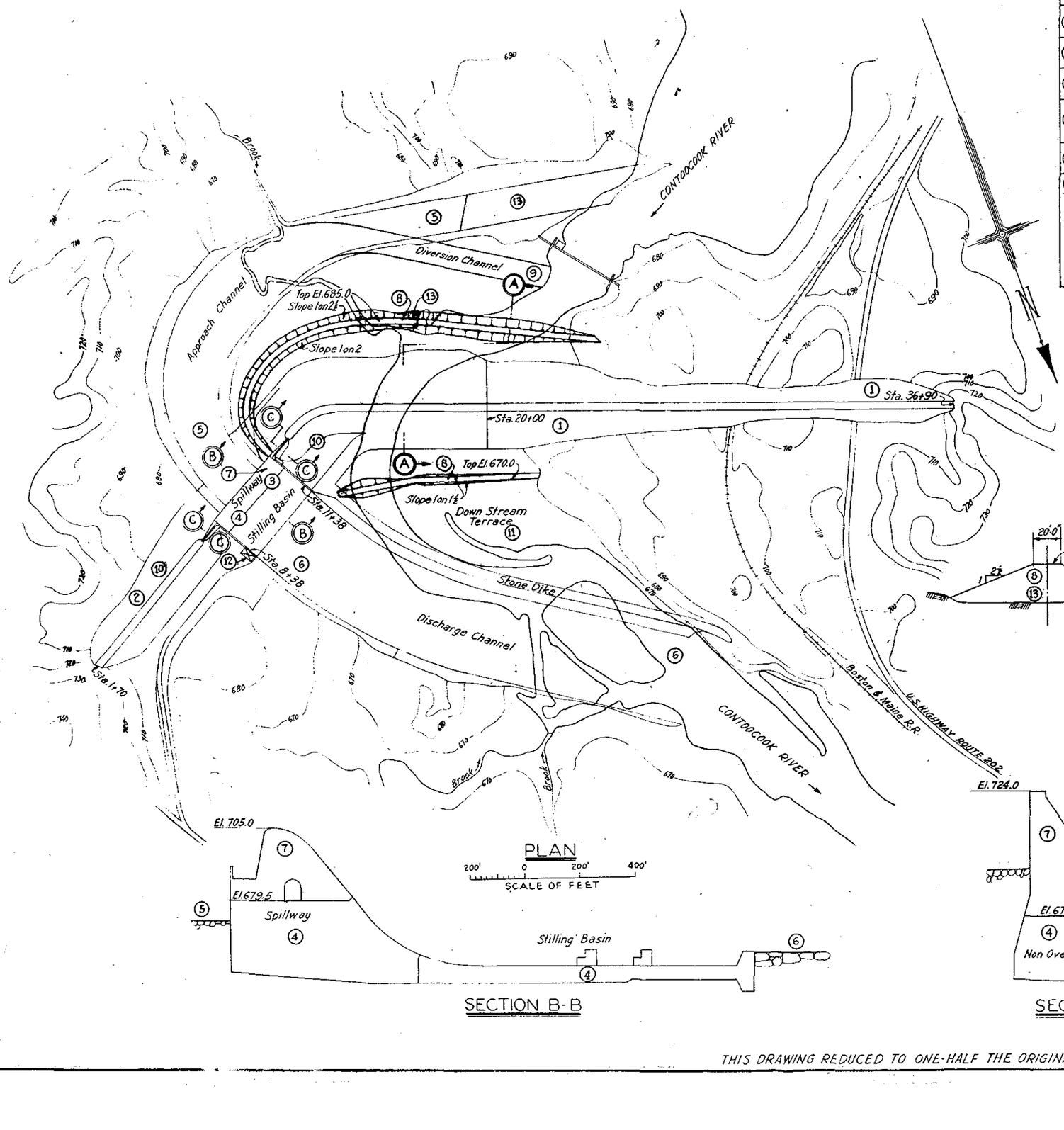
SUBMITTED:
Chief Design Section, Eng. Division

APPROVED:
Division Engineer
District Engineer

DRN. BY JSS(R)
DR. BY Sub-Sect.
TR. BY
CH. BY J.J.S.

FILE NO. M19-13/50

PLATE IV-5



PLAN

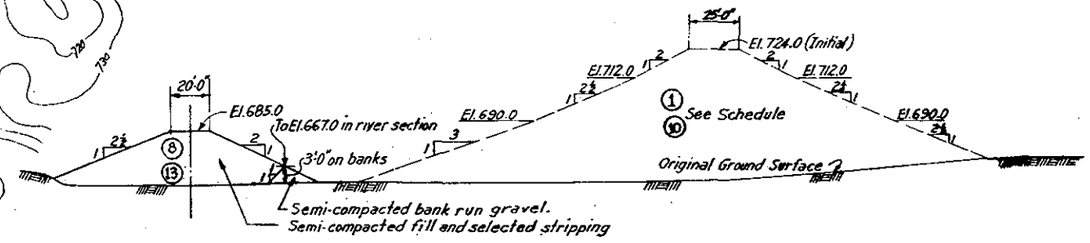
200' 0 200' 400'

SCALE OF FEET

SCHEDULE OF CONSTRUCTION OPERATIONS

FIRST SEASON		SECOND SEASON	
DESCRIPTION OF OPERATIONS	APPROX. QUANTITY CU. YDS.	DESCRIPTION OF OPERATIONS	APPROX. QUANTITY CU. YDS.
(1) Construct section of embankment between Sta. 20+00 and Sta. 36+90, to El. 724.0	EXCAV. = 47,300 FILL = 224,100	(7) Complete Spillway and Non Overflow Sections.	CONC. = 13,600
(2) Construct east embankment to El. 679.5	EXCAV. = 27,100 FILL = 41,000	(8) Construct upstream and downstream cofferdams.	EXCAV. = 700 FILL = 67,200
(3) Excavate area for spillway, stilling basin, non overflow sections and stilling basin walls.	EXCAV. = 115,800	(9) Excavate diversion channel.	EXCAV. = 18,200
(4) Construct Spillway to El. 679.5, Non Overflow Sections to El. 679.5, Stilling Basin and Stilling Basin Walls complete.	CONC. = 39,200 FILL = 13,600	(10) Dewater and construct section of embankment between Sta. 11+38 and Sta. 20+00 to El. 724.0, complete east embankment between Sta. 1+70 and Sta. 8+38.	EXCAV. = 41,400 FILL = 431,600
(5) Excavate approach channel, except a portion at river bank to serve as cofferdam.	EXCAV. = 150,400	(11) Construct downstream terrace and dike	EXCAV. = 12,000 FILL = 138,000
(6) Excavate spillway discharge channel	EXCAV. = 140,300	(12) Construct Equipment House	LUMP SUM
		(13) Remove upstream cofferdam, grade area between approach channel and toe of dam, remove closure cofferdam at entrance to approach channel.	* EXCAV. = 165,300

* Removal of upstream cofferdam amounts to 64,500 cu.yds.

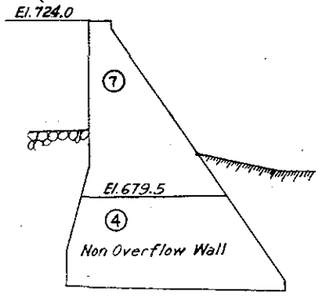


SECTION A-A

40' 0 40' 80'

SCALE OF FEET

SECTION B-B



SECTION C-C

Notes:
Elevations based on U.S.C. & G.S. datum, Mean Sea Level.

20' 0 20' 40'

SCALE OF FEET
(UNLESS OTHERWISE NOTED)

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

NO.	CHARACTER	DATE	APP'D
REVISIONS			

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
SCHEDULE OF CONSTRUCTION OPERATIONS

IN SHEETS SHEET NO.

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL 1951

APPROVAL RECOMMENDED: John S. Allen, Chief Engineering Division

APPROVED: Louis A. Pratt, Colonel, Corps of Engineers, District Engineer

SUBMITTED: F. J. Brown, Chief Design Branch, Div. Engineer

DES. BY ... DET. BY ... CH. OF D.E.R. ... TR. BY ... CH. OF DES. ... CK. BY ...

Peter A. Mueller, Engineer

FILE NO. M19-13/51

PLATE IV-6

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX V

CONSERVATION STORAGE

To accompany definite project report
dated April 1945

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX V - CONSERVATION STORAGE

C O N T E N T S

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PLATES

Plate	Title
V-1	Detailed Cost Estimates
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V-3	River Profile below Bennington Reservoir, N. H., with Existing and Potential Power Developments

DEFINITE PROJECT REPORT
BENNINGTON RESERVOIR
APPENDIX V - CONSERVATION STORAGE

a. General. Multiple-purpose development of the Bennington site for the purposes of flood control and generation of electric power at the site is not feasible due to insufficient undeveloped head. The available head for a distance of one and one-half miles downstream from the proposed dam site is utilized by existing hydro-electric developments which furnish power to local industries that form a substantial economic asset of the area. However, studies of the site have indicated that storage can be obtained at relatively low cost as described in Paragraph w, Appendix I. In view of this, studies were made to determine whether or not it is feasible to provide storage for both flood control and stream regulating purposes at the site.

b. Studies. An inspection of the topographic maps of the reservoir area indicates that the maximum elevation of the water surface is limited by the thickly settled section of Peterboro at the upper end of the reservoir. With this in view, the elevation of the spillway crest for the ultimate installation has been chosen as elevation 712. At this elevation, a reservoir capacity of 90,000 acre-feet can be obtained, of which 40,000 acre-feet would be for conservation storage and 50,000 acre-feet for flood control purposes.

Studies made for period 1920-1940 indicate that the utilization of 40,000 acre-feet of storage at Bennington for stream regulation purposes would provide a regulated flow over the driest period of record of 157 c.f.s.

The developed head of the existing power installations below the Bennington Reservoir total 380 feet of which 210 feet is on the Contoocook River and 170 feet on the Merrimack River (see Plate V-3). The sites on the Contoocook River consist largely of mill-type installations which utilize water power for the direct driving of mechanical equipment and which develop a total of 6,100 H.P. Those on the Merrimack River consist mainly of hydro-electric developments with a total installed capacity of 66,000 KW, most of the head developed being utilized by electric utilities. The improved stream flow will increase the annual energy output of these hydro-electric plants and will also augment their peaking capacities. Furthermore, additional water would be made available at the small plants on the Contoocook River and unsanitary conditions along the rivers would be relieved to some extent. In view of these considerations, a minimum regulated discharge of not less than 100 c.f.s. appears

desirable. A conservative operation for the conservation storage in the reservoir was therefore assumed. This operation is illustrated on the mass diagram for the critical period of record (1930-1931) on Plate V-2. Such operation would increase the downstream low water flow at Manchester from about 1,000 to 1,150 c.f.s. (monthly mean), raising the prime peaking capacity of the existing installations on the Merrimack River by about 2,600 KW. This increase of prime peaking capacity in hydro-electric plants on the Merrimack River could further be increased if the Bennington Reservoir were operated primarily for the benefit of the Merrimack River plants only. No appreciable increase in low water flow in the Contoocook River could then be relied upon.

c. Additional Construction and Real Estate Required for Ultimate Increment. Development of the reservoir for flood control and conservation, if undertaken as a second-stage increment subsequent to the completion of the Flood Control Reservoir, would involve the acquisition of 800 acres of additional land and the raising of 1.5 miles and the relocating of 3 miles of highways. In addition, it would be necessary to acquire one water right. The principal items of construction involved in the second-stage development would include raising the concrete spillway weir 7 feet and the earth embankments and gravity non-over flow sections 6 feet.

d. Basis of Cost Analysis. The evaluation of power benefits to the downstream hydro-electric power plants, produced by the operation of the storage reservoir to increase the low water flows, has been based on the estimated costs for the production of equivalent power by a steam plant located at a load center such as Manchester, New Hampshire. The construction cost of such a steam plant is estimated to be $102 per KW., and the annual costs, consisting of the fixed charges and allowance for operation and maintenance is estimated to be $17.50 per kilowatt per year. This amount is the value assigned per kilowatt of dependable prospective hydro-electric capacity. The value of energy output, based on the current cost of coal fuel at Manchester, is estimated at about 3 mills per kilowatt hour. An allowance of 10% has been made for losses in transmission.

e. Costs. The estimated cost of the project for different storage use and construction stages are tabulated on Plate V-1 and are summarized as follows:

Flood Control Project (Spillway Crest at El. 705)....	$3,886,000.
Flood Control Project, Designed to Permit Future Raising	4,000,000.
Future Raising of Structure and Reservoir.....	1,531,000.
Total Cost, Multiple-Purpose Reservoir.....	$5,531,000.

If the multiple-purpose dam and reservoir were built initially in one-stage construction, the estimated cost would be $5,317,000., which amount would represent a saving of $214,000 over the two-stage construction. It is to be noted that the cost of modifying the flood control structure to permit future raising is $114,000. This added cost, although nonproductive until the structures are raised, represents an investment, the interest charges on which must be met. The total cost of power storage in the amount of $1,645,000. and the corresponding annual charges of $66,977 as shown in Table A, will be subject to an increase to provide for interest which will accrue on the investment of $114,000 required to modify the structures in the initial stage to provide for raising. The amount of this interest is dependent on the length of time elapsing before the second stage of the ultimate project is accomplished.

f. Conclusion. The studies of the benefits to be derived from a multiple-purpose reservoir as shown on Table A fully justify second-stage construction. The ratio of annual benefits to carrying charges for stream regulation storage is 1.12 utilizing all existing power heads downstream and 1.32 utilizing all existing and potential power heads downstream. No value has been claimed for the sanitary and recreational benefits that would accrue from the operation of the multiple-purpose project.

TABLE A

COST ANALYSIS - 2ND STAGE CONSTRUCTION

BENNINGTON RESERVOIR

FOR FLOOD CONTROL AND POWER STORAGE

1.	Reservoir Data		
	Top of Dam	Raised from El. 724 to El. 730	
	Spillway	Raised from El. 705 to El. 712	
	Storage Capacity		Gross
	Flood Control	50,000 A.F.	5.0"
	Conservation	40,000 A.F.	4.0"
	Dead Storage	Pondage to El. 678	-
	Total Storage	90,000 A.F.	9.0"
2.	Estimated Cost of 2nd Stage Construction (incl. Overhead and Contingencies)		
	Dam, Spillway and Outlets	$ 422,000.	
	Reservoir Clearing	375,000.	
	Land and Rights-of-way, incl. Road Relocations	734,000.	
2a.	Total Estimated Cost of 2nd Stage Construction	$1,531,000.	
2b.	Modification of Flood Control Structure to Permit Raising of Dam	$ 114,000.	
	Annual Carrying Charges of Items 2a and 2b	$ 66,977.	

		Head Utilized in Feet	Total Output in KW Hrs.	Cost per KW Hrs. in Mills
3.	Cost per KWH From Storage with annual carrying charges by using:			
	Existing Head below Reservoir	380 ft.	9,800,000	6.8
	Existing and Potential Head below Reservoir	554 ft.	14,300,000	4.7

4.	Annual Power Benefits Downstream (with 80% Water Utilization)			
	Increase in prime peaking capacity	2,600 KW	@ $17.50	$ 45,500.
	Increase in average annual output			
	At existing developed head	9,800,000 KWHS	@ 3 Mills	29,400.
	At existing and potential head	14,300,000 "	@ 3 Mills	42,900.

5a.	Total Annual Power Benefits with Existing Developments	$ 74,900.
5b.	Total Annual Power Benefits with Existing and Potential Developments	88,400.

Ratio: Benefits to Costs for a. with existing developments 1.12

b. with existing and potential developments 1.32

WAR DEPARTMENT — CORPS OF ENGINEERS, U.S.ARMY

		PLAN 1 FLOOD CONTROL ONLY				COMBINED FLOOD CONTROL & CONSERVATION IN 2 STAGES PLAN 2 INITIAL STAGE - FLOOD CONTROL				PLAN 2A INCREMENT FOR COMBINED				COMBINED F.C. & CONSER. IN 1 STAGE PLAN 3 FLOOD CONTROL & CONSERVATION			
	Elevation, Spillway Crest --- Elevation Top of Dam Total Storage Capacity Flood Control Storage Conservation Storage	705.0 — 724.0 60,000 A.F. - Gross 6", Net 8.8" 60,000 A.F.				705.0 — 724.0 60,000 A.F. - Gross 6", Net 8.8" 60,000 A.F.				712.0 — 730.0 90,000 A.F. Gross 9.1" Net 13.2 50,000 A.F. 40,000 A.F. to Elev. 699.5				712.0 — 730.0 90,000 A.F. Gross 9.1" Net 13.2" 50,000 A.F. 40,000 A.F. to Elev. 699.5			
		Quantity	Unit	U.Cost	Cost	Quantity	Unit	U.Cost	Cost	Quantity	Unit	U.Cost	Cost	Quantity	Unit	U.Cost	Cost
I RESERVOIR COSTS	Land and Improvements	—	—	L.S.	$240,500	—	—	L.S.	$240,500	—	—	L.S.	$153,000	—	—	L.S.	$393,500
	Riparian and Water Rights	—	—	L.S.	20,000	—	—	L.S.	20,000	—	—	L.S.	12,000	—	—	L.S.	32,000
	Relocation of telephone and power lines - also highway	—	—	L.S.	655,000	—	—	L.S.	655,000	—	—	L.S.	415,000	—	—	L.S.	1,070,000
	Relocation of railroad	—	—	L.S.	256,000	—	—	L.S.	256,000					—	—	L.S.	256,000
	Sub total				$1,171,500				$1,171,500				$ 580,000				$1,751,500
	Contingencies - 15%				175,725				175,725				87,000				262,725
					$1,347,225				$1,347,225				$667,000				$2,014,225
	Acquisition expenses - 10%				134,775				134,775				67,000				201,775
	TOTAL RESERVOIR COSTS				$1,482,000				$1,482,000				$734,000				$2,216,000
II (a) CONSTRUCTION COSTS	Removal of existing structures	—	—	L.S.	$ 2000	—	—	L.S.	$ 2000	—	—	—	—	—	—	L.S.	$ 2,000
	Stream diversion & pumping	—	—	L.S.	40,000	—	—	L.S.	40,000	—	—	—	—	—	—	L.S.	40,000
	Clearing & grubbing	90	Ac.	$300.00	27,000	90	Ac.	$300.00	27,000	14	Ac.	$300.00	$ 4200	102	Ac.	$300.00	30,600
	Stripping	167,000	C.Y.	0.50	83,500	168,000	C.Y.	0.50	84,000	27,500	C.Y.	0.50	13,750	194,500	C.Y.	0.50	97,250
	Excavation	484,500	C.Y.	0.40	193,800	486,000	C.Y.	0.40	194,400	9,000	C.Y.	0.40	3,600	492,500	C.Y.	0.40	197,000
	Borrow - Impervious	142,400	C.Y.	0.55	78,320	165,000	C.Y.	0.55	90,750	63,000	C.Y.	0.65	40,950	224,000	C.Y.	0.55	123,200
	" - Pervious	353,600	C.Y.	0.50	176,800	350,000	C.Y.	0.50	175,000	96,400	C.Y.	0.60	57,840	438,000	C.Y.	0.50	219,000
	" - Random	118,100	C.Y.	0.50	59,050	110,000	C.Y.	0.50	55,000	101,000	C.Y.	0.55	55,550	218,000	C.Y.	0.50	109,000
	" - Rock	20,000	C.Y.	4.00	80,000	20,000	C.Y.	4.00	80,000	—	—	—	—	3,500	C.Y.	4.00	14,000
	Rolled fill - Impervious	129,000	C.Y.	0.15	19,350	148,000	C.Y.	0.15	22,200	48,500	C.Y.	0.25	12,125	196,000	C.Y.	0.15	29,400
	" " - Pervious	208,000	C.Y.	0.12	24,960	200,000	C.Y.	0.12	24,000	41,500	C.Y.	0.17	7,055	249,500	C.Y.	0.12	29,940
	" " - Random	203,000	C.Y.	0.12	24,360	195,000	C.Y.	0.12	23,400	40,500	C.Y.	0.17	6,885	235,500	C.Y.	0.12	28,260
	" " - Semi-compacted	175,000	C.Y.	0.10	17,500	177,500	C.Y.	0.10	17,750	37,000	C.Y.	0.15	5,550	213,000	C.Y.	0.10	21,300
	Structure backfill	18,500	C.Y.	0.60	11,100	18,500	C.Y.	0.60	11,100	—	—	—	—	18,500	C.Y.	0.60	11,100
	Screened gravel backing	30,000	C.Y.	2.00	60,000	29,500	C.Y.	2.00	59,000	6,000	C.Y.	2.25	13,500	31,000	C.Y.	2.00	62,000
	Filter sand and gravel	48,000	C.Y.	1.30	60,400	48,000	C.Y.	1.30	62,400	9,500	C.Y.	1.50	14,250	57,000	C.Y.	1.30	74,100
	Gravel facing	8,500	C.Y.	1.25	10,625	8,500	C.Y.	1.25	10,625	8,800	C.Y.	1.50	13,200	8,800	C.Y.	1.25	11,000
	Dumped rip rap	86,500	C.Y.	0.60	51,900	85,500	C.Y.	0.60	51,300	13,300	C.Y.	0.60	7,980	90,500	C.Y.	0.60	54,300
	Derrick stone	5,000	C.Y.	5.00	25,000	5,000	C.Y.	5.00	25,000	—	—	—	—	5,000	C.Y.	5.00	25,000
	Road surfacing	2,700	S.Y.	0.30	810	2,700	S.Y.	0.30	810	—	—	—	—	2,700	S.Y.	0.30	810
	Concrete - Spillway, Still. Basin & Non-overflow	40,000	C.Y.	13.50	540,000	45,000	C.Y.	13.50	607,500	2,700	C.Y.	16.00	43,200	47,500	C.Y.	13.50	641,250
	" - Stilling Basin guide walls	7,800	C.Y.	15.00	117,000	7,800	C.Y.	15.00	117,000	—	—	—	—	7,800	C.Y.	15.00	117,000
	Reinforcing steel	476,000	Lb.	0.06	28,560	595,000	Lb.	0.06	35,700	59,000	Lb.	0.06	3,540	645,000	Lb.	0.06	38,700
	Well system	—	—	L.S.	24,000	—	—	L.S.	24,000	—	—	—	—	—	—	L.S.	24,000
	Equipment house & Operators quarters	—	—	L.S.	25,000	—	—	L.S.	25,000	—	—	—	—	—	—	L.S.	25,000
	Misc. metals, Trash bars, Emerg. gates & Monorail	—	—	L.S.	13,000	—	—	L.S.	13,000	—	—	L.S.	4,100	—	—	L.S.	15,100
	Gates and Hoists	—	—	L.S.	60,000	—	—	L.S.	60,000	—	—	—	—	—	—	L.S.	60,000
	Lighting & Power system	—	—	L.S.	15,000	—	—	L.S.	15,000	—	—	—	—	—	—	L.S.	15,000
	Oil pressure system & Misc. equipment	—	—	L.S.	10,000	—	—	L.S.	10,000	—	—	—	—	—	—	L.S.	10,000
	Removing & cleaning concrete	—	—	—	—	—	—	—	—	—	—	L.S.	15,000	—	—	—	—
	Miscellaneous items	—	—	L.S.	38,965	—	—	—	48,065	—	—	L.S.	14,925	—	—	L.S.	55,690
	Sub Total				$1,920,000				$2,011,000				$ 337,200				$2,181,000
	Engr. Inspection, Overhead & Contingencies (25%)				480,000				503,000				84,800				545,000
	TOTAL CONSTRUCTION COSTS				$2,400,000				$2,514,000				$ 422,000				$2726,000
II (b) RESERVOIR CLEARING	Flood Control	—	—	L.S.	$ 3,000	—	—	L.S.	$ 3,000	—	—	—	—	—	—	—	—
	Conservation	—	—	—	—	—	—	—	—	—	—	L.S.	$ 300,000	—	—	—	$ 300,000
	Sub Total				$ 3,000				$ 3000				$ 300,000				$ 300,000
	Engr. Inspection, Overhead & Contingencies (25%)				1,000				1,000				75,000				75,000
	TOTAL RESERVOIR CLEARING				$ 4,000				$ 4,000				$ 375,000				$ 375,000
TOTAL ESTIMATED COST					$3,886,000				$4,000,000				$1,531,000				$5,317,000
COST PER ACRE FOOT		$64.77				$66.67				* $51.03				$59.08			

* Cost per Acre Foot is based on 30,000 Acre Feet of Increased Storage.

Drainage area { Gross - 186 sq. mi. / Net - 128 sq. mi.

MERRIMACK VALLEY FLOOD CONTROL

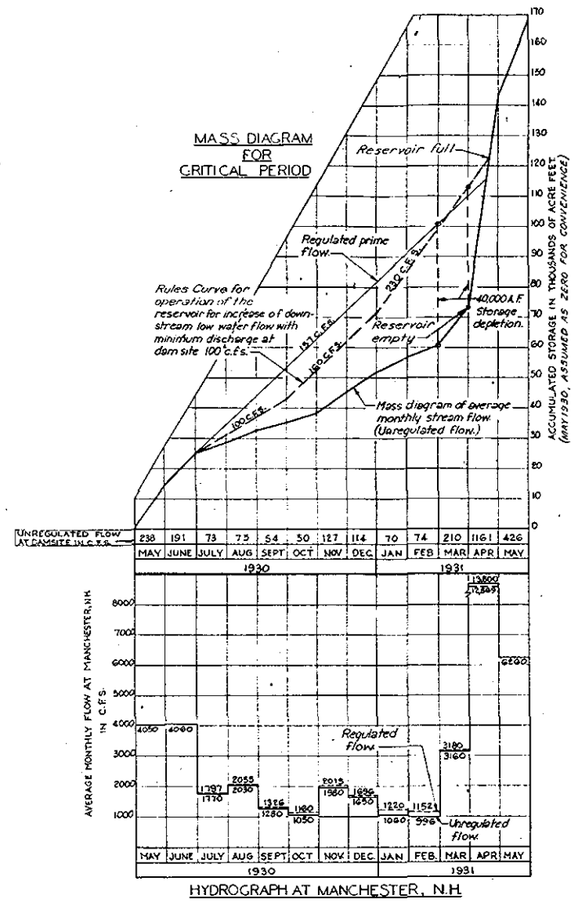
BENNINGTON DAM
CONTOOCOOK RIVER

DETAILED COST ESTIMATES

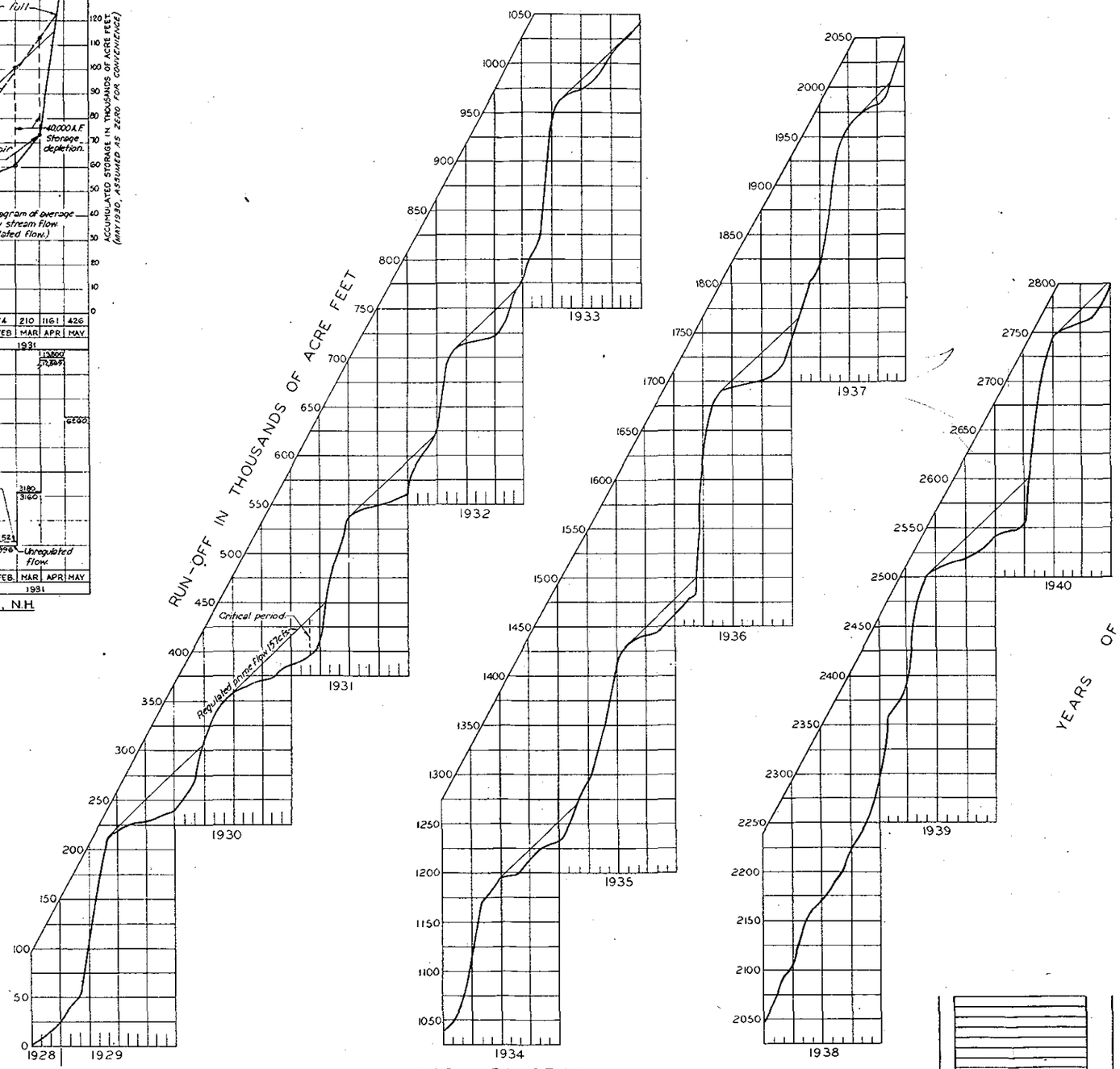
U.S. ENGINEER OFFICE, BOSTON, MASS.

FILE NO. M19-13/52 APRIL 1945

PLATE V-1



HYDROGRAPH AT MANCHESTER, N.H.



MASS DIAGRAM

Notes:-

1. Drainage area at Bennington dam site:-186 square miles.
2. Monthly unit flows used are averages of those recorded at the U.S.G.S. gaging stations at:-
   Penacook, N.H. (766 Square miles)
   Blackwater, N.H. (134 Square miles)
   Antrim, N.H. (54.8 Square miles)
3. The mass diagram shown represents average monthly stream flow at the dam site with deduction of evaporation and seepage losses.
4. These data apply to a combined development of the site for flood control and conservation:
   Usable conservation storage- 40,000 A.F.-El.678 to El.699.5.
   Flood control storage - 50,000 A.F.-El.699.5 to El.712.
5. Rules Curve for operation of the reservoir is illustrated on mass diagram for critical period.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER
FLOW DATA
CONSERVATION STORAGE

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL, 1945

APPROVAL RECOMMENDED	APPROVED:
John E. Allen, Chief Engineering Division	Thomas A. Cuff, Colonel, Corps of Engineers, District Engineer
SUBMITTED: Chief Design Section, Engineering Division	
Fred J. Nebiker, Engineer	FILE NO. M19-13/53

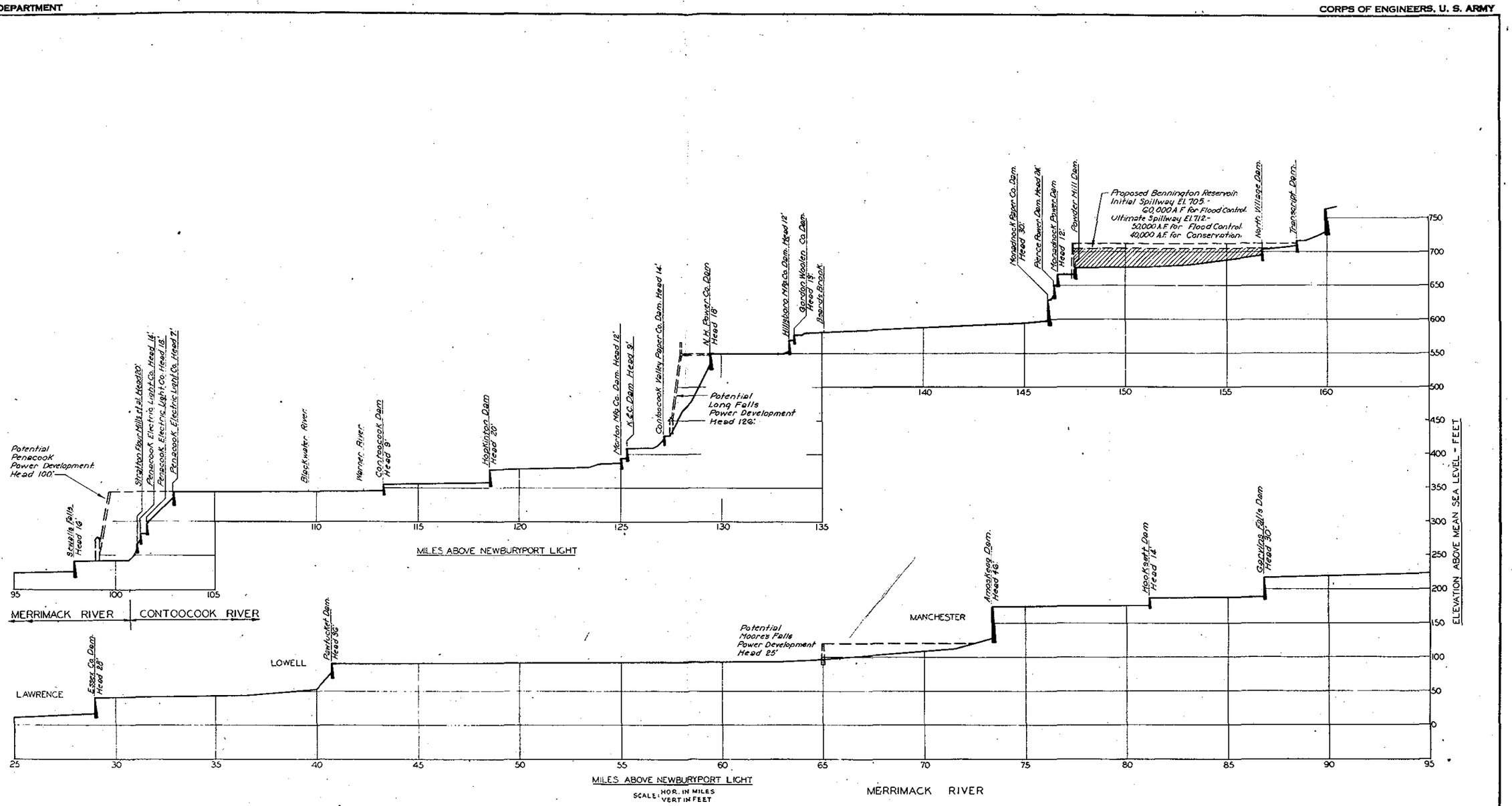
NO.	CHARACTER	DATE	BY
	REVISIONS		

DES. BY
DR. BY
TR. BY
CH. BY

PLATE V-2

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL

RIVER PROFILE BELOW BENNINGTON, RESERVOIR, N.H. WITH EXISTING AND POTENTIAL POWER DEVELOPMENTS

U. S. ENGINEER OFFICE, BOSTON, MASS. 18 APRIL, 1945

APPROVAL RECOMMENDED

Chief Engineering Division

APPROVED:

Colonel, Corps of Engineers
District Engineer

SUBMITTED:

Chief District Section, Engineering Division

Asst. Engr.

FILE NO. M 19-13/54

NO.	CHARACTER	DATE	BY
	REVISIONS		

PLATE V-3

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX VI

RELOCATIONS

To accompany definite project report
Dated April 1945

DEFINITE PROJECT REPORT

BENNINGTON RESERVOIR

APPENDIX VI - RELOCATION

C O N T E N T S

Paragraph	Title	Page
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c.	Cemeteries	VI-3
d.	Pipe and Water Supply Lines	VI-3
e.	Power, Telephone, and Telegraph Lines	VI-3
f.	Water Rights	VI-3
g.	Method of Accomplishing Relocations	VI-4
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PLATES

Plate	Title
VI-1	Railroad and Highway Relocation
VI-2	Boston & Maine Service and Connections

DEFINITE PROJECT REPORT
BENNINGTON RESERVOIR

APPENDIX VI - RELOCATIONS

a. Railroads.- At the present time, a branch line of the Boston & Maine Railroad is located within the proposed reservoir basin, as shown on Plates VI-1 and VI-2, accompanying this appendix. This line, originating at Nashua, New Hampshire, runs through the reservoir area from Greenfield to Elmwood to Bennington, and then extends as far as Hillsboro. This branch previously looped around and connected at Contoocook, N. H., with the main line extending from Claremont Junction to Concord. A portion of the loop between West Henniker and Hillsboro, N. H., was washed out during the 1936 Flood and never repaired. Therefore, those towns from West Henniker north are now serviced by that portion of the loop that ties into the main line at Contoocook, N. H. The general arrangement of these railroad lines is shown on the map of Boston and Maine Services and Connections, Plate VI-2.

A number of layouts have been made in conjunction with the Boston & Maine Railroad, and estimates prepared for relocating that portion of the road, Greenfield-Elmwood-Bennington, which would be within the inundated areas, upstream from the proposed dam. The possible relocations that were studied did not prove satisfactory to the Boston & Maine Railroad as the minimum grade that could be obtained was too steep and furthermore the cost estimates of the proposed relocations were prohibitive due to the fact that a bridge and considerable fill in order to obtain a minimum grade would be required. It has been estimated by the Boston & Maine Railroad that the cost of relocation at the dam site would be approximately $918,000.

As an alternate to its relocation, it has been proposed to have the railroad abandon the line from Greenfield through Elmwood to Bennington and to rehabilitate the line between West Henniker and Hillsboro which was destroyed in 1936 as indicated above. In this way, service would be continued to all of those localities now served by the railroad. Representatives of the railroad have informally agreed to this plan, although the rail distance from Boston to Bennington will be about 35 miles greater than over the present lines. It is estimated that the cost of accomplishing the above noted rehabilitation, and the cost of removal of the existing line within the reservoir area is $256,000.

The method of railroad abandonment and relocation is subject to the approval of the Interstate Commerce Commission. However, the matter has not yet been referred to the Commission, and therefore no information can be furnished as to its opinion on the above proposal.

The railroad lines shown on Plates VI-1 and VI-2, extending from Elmwood south to Peterboro and from Elmwood west toward Keene have been abandoned, and the only expense to the Government in acquiring the right-of-way would be the cost of the land.

b. Highways and Roads.- There is one main highway and some secondary roads that will be affected by construction of the proposed reservoir and that will require relocation or raising. These roads and the proposed relocations are indicated on the map accompanying this appendix Plate VI-1.

U. S. Highway 202, a first-class highway connecting Peterboro and Bennington, N. H., traverses along the west side of the river and is subject to inundation for a distance of 2.8 miles in the vicinity of Nahors and for a distance of 1.5 mile immediately upstream from the dam site. It is proposed to relocate these portions of the highway on high ground above elevation 715 on the westerly side of the reservoir so that no further raising would be required for the proposed ultimate development.

The existing steel and concrete bridge on Route 202 at North Village was recently reconstructed and is in excellent condition. This bridge has sufficient clearance for the flood stage of the initial development and no alterations are required until such time as the ultimate development is undertaken.

The present second class surface-treated highway on the easterly side of the dam site must be relocated where it passes through the proposed dam site and also where it would be inundated immediately upstream from the dam. It is proposed to relocate the portion at the dam site on high ground to the east of its present site. The remainder of this road skirting the reservoir would be raised to elevation 708, where required, for the initial development. Those portions of the road that would be inundated after construction of the ultimate stage would be raised to elevation 715 or relocated at the time of the ultimate construction.

The road from Greenfield to Hancock crosses directly through the reservoir basin and is a narrow gravel road. In order to raise this road above the flood stage, considerable fill would be required and the existing wooden covered bridge would require raising and new abutments. The amount of traffic now using this road does not warrant the expenditure of large funds to raise it above the high water level. Therefore, it is proposed to raise the road and bridge to elevation 695 within the proposed reservoir basin, so as to maintain uninterrupted traffic during the periods of high water other than extreme high flood stages, and when the ultimate project is undertaken to relocate the road to elevation 715 on the high ground just downstream from its present site, and construct a new bridge over the river.

It is not proposed to raise the existing river crossing of the secondary road and bridge 1.5 miles north of Nahors as traffic can still use this road under normal conditions, and use the peripheral roads during high water stages.

The Chief Engineer of the Highway Department, State of New Hampshire, has been consulted and has agreed that the relocations proposed are a reasonable solution for raising and relocating the network of roads affected by construction of the proposed dam.

c. Cemeteries.- There are no cemeteries within the proposed reservoir basin of either the initial or ultimate developments. However, there are two locations at which there is evidence that the areas might have been used as private burying grounds.

d. Pipe and Water Supply Lines.- There are no major pipe or water supply lines within the area of the proposed reservoir basin of either the initial or ultimate developments.

e. Power, Telephone and Telegraph Lines.- There is a trunk telephone line passing through the reservoir area that will require relocating. There are also telephone, telegraph and power lines within the area that provide local service only, and which in general, will no longer be needed when the reservoir is constructed. No high tension transmission lines exist within the proposed reservoir area of either the initial or ultimate developments.

f. Water Rights.- In the initial flood control development, there will be four water rights affected by construction of the reservoir. These are the rights connected with the developments at the Powder Mill Dam, located just southerly of the site of the proposed dam, the Bell Dam, situated at North Village, the Simonds Dam located northwesterly of Happy Valley, and a dam on Ferguson Brook that is used to develop power for a small saw mill. In the ultimate development, the Transcript Dam at Peterboro will also be affected.

The major one of these rights is the Powder Mill Dam which is owned and operated by the Monadnock Paper Mills, Bennington, N. H., and is located approximately 800 feet upstream from the proposed flood control dam. This dam provides storage water for the commercial manufacture of paper and generation of power by the paper mill which is located in the village of Bennington. Several conferences have been held with the owner of the Monadnock Paper Mills on the construction of the proposed reservoir which will inundate the Powder Mill Dam. The owner has consented in writing to the location of the proposed dam, provided that the Government will assure him the same amount of water storage as he now controls with the

Powder Mill Dam, for his use after construction of the flood control dam and provided that water will be released from storage in a manner that will meet the requirements of the paper mill. The owner has further stipulated that he would hold the Government liable for any loss of power or curtailment in manufacture as a result of the construction of the dam. The owner will not consent to the purchase of the water rights as a whole. Under the present schedules of operations it is anticipated that the only time the mills will be curtailed in the use of water is the short period of time during the construction of the cofferdams and the opening up of the diversion channel. The operations and maintenance schedule provides for control of the gates and pond in a manner to assure the owner of the mills the same water rights he now enjoys.

The other existing water rights pertain to minor installations and will be obtained by direct negotiation with the owners.

g. Method of Accomplishing Relocations.- Where it is necessary to relocate utilities it is proposed to have the work accomplished by the respective owner through contractual arrangements.

h. Source of Information.- In accordance with the requirements of Circular Letter No. 3570, Real Estate No. 62, dated 21 February 1945, subject, "Real Estate Functions of Division Offices," the method of disposing of utilities lying in the reservoir area was developed in collaboration with representatives of the Real Estate Division of the Office of the Division Engineer, New England Division. The general description of the real estate involved and data as to its acquisition cost were taken from the "Report - Real Estate Cost, Bennington Reservoir, N. H.," dated 21 April 1945, prepared by the Division Engineer, New England Division.

ABANDONED RAILROAD
(HILLSBORO TO EMERSON STA.)
TO BE REHABILITATED TO MAINTAIN SERVICE TO HILLSBORO AND BENNINGTON WHEN THE RESERVOIR IS CONSTRUCTED.

WIDEN EXISTING HIGHWAY

HIGHWAY RELOCATION
(MIN. EL. 715.0)

BENNINGTON DAM

ABANDONED RAILROAD
(ELMWOOD TO KEENE)

HIGHWAY RELOCATION
(MIN. EL. 715.0)

EXISTING RAILROAD *
(ELMWOOD TO BENNINGTON)
(TO BE ABANDONED)

HIGHWAY RELOCATION
(FOR ULTIMATE DEVELOPMENT)

RAISE ROAD
(MIN. EL. 708.0)

EXISTING RAILROAD *
(ELMWOOD TO GREENFIELD)
(TO BE ABANDONED)

HIGHWAY RELOCATION
(FOR ULTIMATE DEVELOPMENT)

ABANDONED RAILROAD
(ELMWOOD TO PETERBORO)

RAISE ROAD
(MIN. EL. 695.0)

FLOW LINE, EL. 705.0

HIGHWAY RELOCATION
(MIN. EL. 715.0)

RAISE ROAD
(MIN. EL. 708.0)

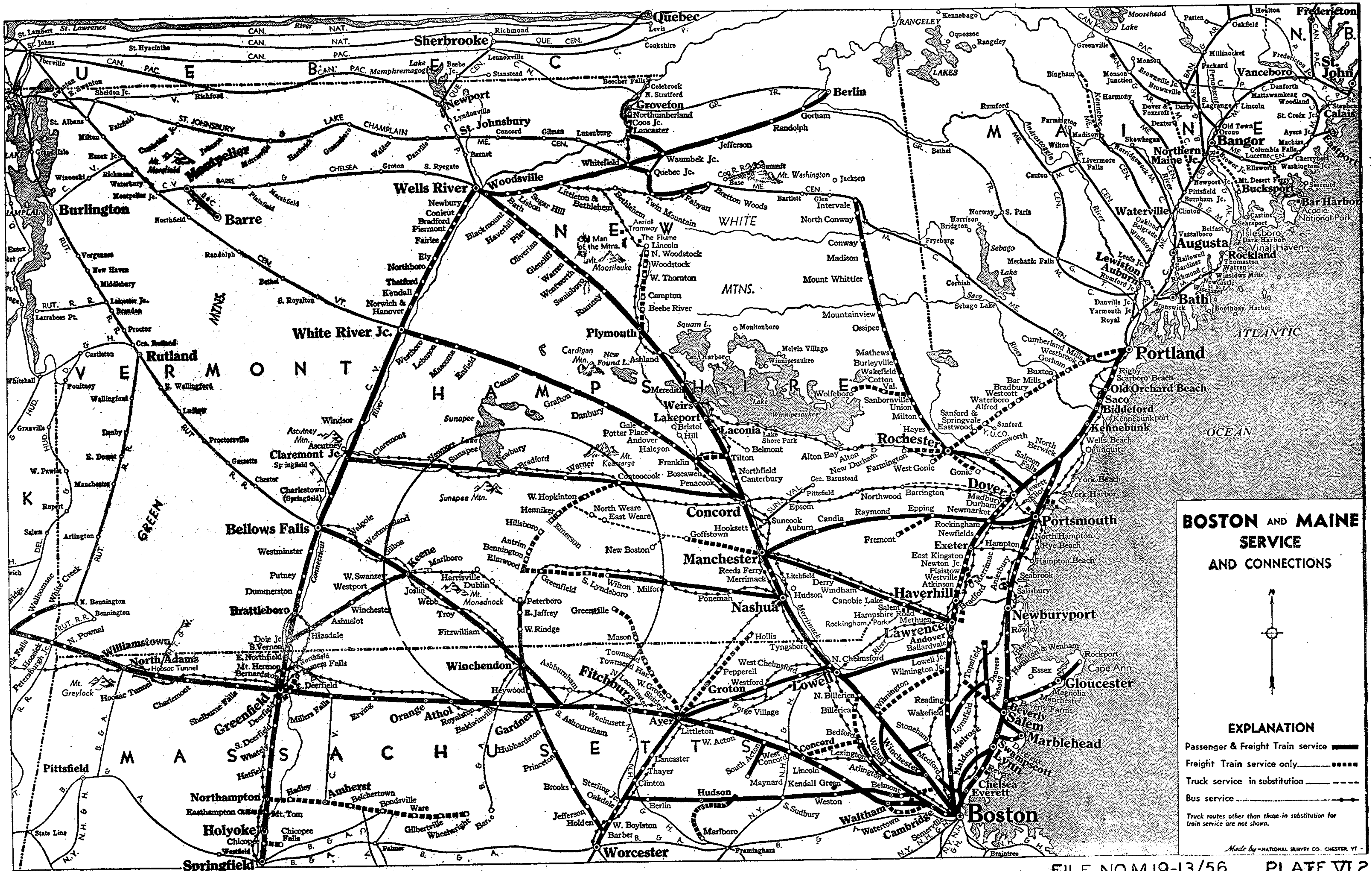
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
RAILROAD & HIGHWAY
RELOCATION

U. S. ENGINEER OFFICE, BOSTON MASS.
APRIL, 1945
FILE NO 19-13/55

PLATE VII-1

NOTE
IN VIEW OF INACCURACY OF THE U.S.G.C. CONTOURS IN SOME PARTS OF THE RESERVOIR AREA, THE FLOW LINE IS PLOTTED ON THE BASIS OF CONTOURS OBTAINED FROM AERIAL TOPOGRAPHIC SURVEY.

* SERVICE TO BENNINGTON AND HILLSBORO TO BE MAINTAINED AFTER CONSTRUCTION OF RESERVOIR, BY REHABILITATION OF PRESENTLY ABANDONED LINE FROM HILLSBORO TO EMERSON STA., AS SHOWN ON THIS PLAN.



FILE NO. M 19-13/56 PLATE VI 2